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A COMPUTER - AIDED SYSTEM FOR THE ANALYSIS, DESIGN AND CHECKING OF CONCRETE STRUCTURES

by

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NOTATION

A	= Cross-sectional area of member
A_s	= Area of tensile steel
A'_s	= Area of compressive steel
A_{st}	= Total area of longitudinal steel in columns
D_{ij}	= Deformations of story i due to loading j
d	= effective depth of section
d_t	= Total depth of a slab
d_{bc}, d_{tc}	= Bottom and top clear covers for slab steel
d_{cc}	= Clear cover for column reinforcement
e	= Eccentricity of axial force
E	= Modulus of elasticity
f_c	= Extreme fiber compressive stress in concrete
f_{ca}	= Allowable stress in concrete
f'_c	= Compressive strength of concrete
f_s	= Tensile steel stress
f'_s	= Compressive steel stress
f_{sa}	= Allowable steel stress
f_y	= Yield strength of reinforcing steel
G	= Shearing modulus
H	= Translation matrix from end B to end A of member
I_y	= Moment of Inertia of cross-section with respect to y-axis
I_z	= Moment of inertia of cross-section with respect to z-axis
J	= Torsional constant for cross-section
K^*_m	= Stiffness matrix of member with respect to local axes

$K^*_{AA}, K^*_{AB}, K^*_{BA}, K^*_{BB}$ = Stiffness submatrices of member with respect to local axes

K_m = Stiffness matrix of member with respect to global axes

$K_{AA}, K_{AB}, K_{BA}, K_{BB}$ = Stiffness submatrices of member with respect to global axes

k'_{jj} = Stiffness submatrix of joint j

k'_{jg} = Cross-stiffness submatrices between joint j and its neighboring joints

k_j = Cross-stiffness submatrix between joint j and corresponding upper floor joint

K_{ii} = Direct stiffnesses of substructures (floor stiffness matrices)

$K_{i,i-1}, K_{i,i+1}$ = Cross-stiffnesses between floor i and adjacent floors

K_0 = Modified joint stiffness matrix

L = Length of member

M_A, M_B = Girder end moments

M_c = Midspan moment of girder

M_x, M_y = Bending moments of column about X and Y principal axes

M_r = Ultimate resisting moment of cross-section

n = Number of stories

P = Column axial force

P_{ij} = Joint load vector of story i due to loading j

P_j = Actions at joint j

P_b = Balanced load for a column

P_0 = Load capacity of a column in pure axial compression

P_{ult} = Ultimate strength of a column

p = Reinforcement ratio

R = Column strength reduction factor

r = Number of joints per floor

T = Rotation matrix from member to global coordinates

t = Side dimension of a column

X,Y,Z = Global coordinate axes

x,y,z = Local coordinate axes

$\delta_{j1} \ldots \delta_{j6}$ = Displacements of joint j

CHAPTER 1

INTRODUCTION

1.1 Object of Study

The phenomenal impact and influence of electronic computers on present-day technology has already been felt in most areas of structural engineering. Numerous structural analysis and design programs have been developed, including some large systems based on problem-oriented languages. However, due to the nature of most available computers, the vast majority of existing programs are geared to a batch processing environment.

The presently emerging third generation computers, however, make many earlier program limitations unnecessary. Control, communication, and programming facilities for direct, interactive man-machine communication are now being rapidly developed. In order to benefit fully from these developments, it is imperative that the possibilities, application programming requirements, and implications inherent in this new environment be investigated from the standpoint of their application to structural engineering. It is, therefore, felt that the development of computer-aided systems for the analysis, design and checking of structures in the light of present and projected computer facilities is of great significance.

The goal of this study is to investigate and develop a pilot system for this class of applications.

1.2 Scope of Study

The development of a completely general system for the analysis, design and checking of any type of structure is a subject much too large for an individual study. On the other hand, it is not possible to consider the programming and decision-making needs without involving the layout, geometry, material properties, etc. of a specific class of structures. For this reason, the present study deals with only one type of structure.

The type of structure selected for this study is a flat plate reinforced concrete building, consisting solely of floor panels of uniform depth and square tied columns of uniform cross-section. The geometry of the building is assumed to be regular, and the slabs are idealized as girders of one panel width each in order to facilitate an elastic frame analysis of the structure. The design of the slabs and columns is performed by means of the working stress or ultimate strength design methods in accordance with the current ACI Code (1)* specifications.

Throughout the report, a distinction is being made between the terms "model" and "system". As used herein, the term model refers to the set of algorithms which perform specific structural calculations, whereas the term system refers to the combination of algorithms used to perform distinct analysis, design and checking tasks.

It is recognized that in the total building process, the analytical and data-processing capabilities of the model are used in two distinct ways: first, by the agency called the "designer" responsible for the creation of the structural configuration, and, secondly, by the agency called the "checker" responsible for ascertaining that the designer's product satisfies the legal requirements embodied in the

* Numbers in parentheses refer to entries in List of References.

design specifications. The system described in this report not only allows for both of these functions, but also provides for the efficient transmission of the necessary information from the designer to the checker.

Although not specifically incorporated in the study, consideration has also been given to the third agency involved in the building process, namely the contractor responsible for building the structure. The data describing the structure are in a form which can be efficiently used as input data to a contractor's estimating, planning, and scheduling programs.

1.3 Computer-Aided Design

The design of a structure is a complex problem, which calls for the judgment and decision-making capabilities of a competent engineer throughout the design process. As opposed to purely computational processes, many of the decision-making problems encountered during the design of a structure are qualitative, rather than quantitative, in nature.

The need for qualitative decisions during the design process renders it extremely hard, and probably futile as well, to attempt to automate completely the entire design of a structure. What is required is a much more flexible system, which allows for the engineer's participation at all the decision-making stages in the process. Thus, for instance, the system must be designed such that it allows the engineer to modify the input data or suggest a particular procedure after examining some intermediate results of the design process. Such a system is considered to be computer-aided rather than automated.

In the context of the present study, it is recognized that the designer deals with problems other than the analysis and member proportioning of a fixed structural configuration. Similarly, the checker must also consider factors other than the adequacy of the structural elements as defined by the specifications. However, by relegating these processes to the digital computer, the present system can serve as a valuable tool for both agencies.

1.4 Organization of Report

Chapter 2 describes the analysis and design model incorporated into the computer-aided system developed in this study. The type of structure to which the model is applicable is described first. Next, the alternative methods of analysis, and the types of loads and loading combinations incorporated in the model are outlined. This is followed by a description of the member design procedures and the design aids included in the model. The application of the model to the checking of a structure and evaluation of the material quantities are also discussed.

Chapter 3 complements Chapter 2 and contains a detailed description of the mathematical procedures employed in the implementation of the model.

Chapter 4 is devoted to an examination of numerical results obtained using the model. A number of sample problems were considered, and the results provided by the alternative methods of analysis and design are tabulated and compared.

In Chapter 5 the present organization of the system is described, and the various applications of the system are discussed. In Chapter 6, the applications of the system are illustrated by several examples, and the pertinent results are presented.

Chapter 7 contains conclusions reached during the development and testing of the model and the system. Several possible extensions of the present study are also suggested.

CHAPTER 2

DESCRIPTION OF THE MODEL

The analysis and design model consists of a set of algorithms or procedures which resemble closely the methods normally employed by a designer during the manual analysis and design of a structure. The algorithms incorporated in the model attempt to formalize the various alternatives and empirical relations routinely used by designers. This chapter describes the model and its capabilities, which include analysis, design, checking and quantity take-off. The available alternative methods of analysis, acceptable types of loads, and member design methods are outlined. A detailed description of the mathematical procedures employed in the implementation of the model is deferred until the following chapter.

2.1 Choice of Structure

The model developed in this study is applicable to the analysis and design of flat plate reinforced concrete buildings. The structure is assumed to consist solely of floor panels of uniform depth and square tied columns of uniform cross-section. As a further simplification, the layout of the buildings is assumed to be completely regular with no missing slabs or columns, as shown in Fig. 2.1. The limitation of regularity covers only the configuration of the structure, not the dimensions. Thus all individual bays, aisles and stories may have different dimensions.

It is recognized, of course, that the assumption of a regular structure is not always valid for practical structures. However, this disadvantage is offset by the fact that for a regular layout all the topological relations and geometrical properties of the structure required during the processing can be generated internally. For a regular layout, the number of bays, aisles and stories in the structure completely define the topology of the structure, whereas the bay and aisle widths and story heights are sufficient to derive the geometrical properties of the structure. For an irregular building layout, all the topological relations and geometrical data required for the solution would have to be provided as input data.

2.2 Analysis

The function of the analysis algorithm is to provide several options for the analysis of the structure. The alternatives incorporated into the model permit a range of analyses which strike a balance between the accuracy and economic requirements of the designer.

2.2.1 Method of Analysis

As recommended in Section 2102 of the ACI Code, the floor slabs are idealized as longitudinal and lateral girders of one panel width each, and the structure is treated as an elastic frame. The shaded portion of the floor slab in Fig. 2.1, for instance, comprises a longitudinal girder.

The elastic frame analysis is performed by the stiffness method, and the idealized structure may be analyzed either as a space frame or as a series of plane frames. Furthermore, in order to provide the desired flexibility in the choice of the method of analysis, several levels of accuracy are included within the space and plane frame analyses.

2.2.2 Alternatives in Space Frame Analysis

A space frame joint has six degrees of freedom, which consist of three translations and three rotations about the coordinate axes, as shown in Fig. 2.2(a). In the stiffness method, the displacements of the structure are the generalized unknowns, and, consequently, the processing time for a problem increases rapidly with the number of degrees of freedom of the structure. Hence, the computer time required for the analysis of a large structure may easily become prohibitive, especially when a designer requires only an approximate analysis, or feels that certain deformation components may be neglected without seriously affecting the accuracy of the solution. In an attempt to provide for such situations, ten different alternatives have been built into the analysis algorithm. These alternatives allow the designer to neglect certain deformations and range from allowing all six deformations at a joint to allowing only the two bending rotations at each joint. The alternatives are as follows:

ALTERNATIVE 1: Allow all six degrees of freedom at a joint (Fig. 2.2(a)).

ALTERNATIVE 2: Neglect the axial deformations of the columns. This reduces the number of degrees of freedom to five per joint (Fig. 2.2(b)).

ALTERNATIVE 3: Neglect the rotation of the joints about the vertical axis (Fig. 2.2(c)). This is equivalent to neglecting the in-plane twisting of the floors.

ALTERNATIVE 4: Neglect the axial deformations of the girders. This assumption causes the lateral displacements of all joints in a frame at each floor level to be equal, so that there is only one unknown lateral displacement per frame per floor level. Figure 2.2(d) shows the resulting lateral degrees of freedom of a typical floor. The other four degrees of freedom at each joint, namely the vertical translation and the three rotations, are unaffected and remain distinct at each joint.

ALTERNATIVE 5: Neglect the transverse deformations due to twisting of the girders. This assumption causes the lateral displacements perpendicular to a frame at all joints in the frame at each floor level to be equal. Moreover, since the lateral displacements perpendicular to the frame are equal at the ends of each girder, there cannot be any relative rotation about the vertical axis between girder ends. This means that the rotation about the vertical axis is equal at every joint on a floor level, so that there is only one unknown rotation about the vertical axis per floor level. The degrees of freedom of a typical floor are shown in Fig. 2.2(e).

ALTERNATIVE 6: Neglect the axial and transverse deformations of the girders. This assumption implies that the floor slabs act as rigid bodies with regard to all in-plane deformations. Hence, the lateral displacements and the rotation about the vertical axis are equal at every joint on a floor level, resulting in only three in-plane rigid body displacements per floor as shown in Fig. 2.2(f). This alternative corresponds to the "tier building" formulation presented by Weaver.[8]

ALTERNATIVE 7: Neglect the axial deformations of both the girders and columns. This is a combination of Alternatives 2 and 4.

ALTERNATIVE 8: Neglect the transverse deformations of the girders and the axial deformations of the columns. This is a combination of Alternatives 2 and 5.

ALTERNATIVE 9: Neglect the axial deformations of the columns as well as the axial and transverse deformations of the girders. This is a combination of Alternatives 2 and 6.

ALTERNATIVE 10: Consider only the bending rotations of the joints. This results in only two degrees of freedom per joint of the structure as shown in Fig. 2.2(g). Because lateral displacements are neglected, this alternative is applicable to vertical loads only.

2.2.3 Alternatives in Plane Frame Analysis

As mentioned before, the structure may also be analyzed as a series of longitudinal and lateral frames of one panel width each. Typical longitudinal and lateral frames are shown in Fig. 2.1. Every joint of a plane frame has three degrees of freedom, which consist of a vertical translation, a lateral translation and a bending rotation. In keeping with the stated objectives, six alternative plane frame analysis options are incorporated into the model. These alternatives provide a range for the degrees of freedom from three degrees of freedom per joint to one degree of freedom per floor. The alternatives are as follows:

ALTERNATIVE 1: Allow all three degrees of freedom at a joint (Fig. 2.3(a)).

ALTERNATIVE 2: Neglect the axial deformations of the columns. This assumption reduces the number of degrees of freedom to two per joint (Fig. 2.3(b)).

ALTERNATIVE 3: Neglect the axial deformations of the girders (Fig. 2.3(c)).

ALTERNATIVE 4: Neglect the axial deformations of both the girders and columns (Fig. 2.3(d)). This alternative corresponds to the "classical" rigid frame analysis.

ALTERNATIVE 5: Consider only the bending rotation of each joint (Fig. 2.3(e)). Since sidesway is neglected, this alternative is applicable to vertical loads only.

ALTERNATIVE 6: Neglect all the axial deformations and rotations, considering only the sidesway of the structure (Fig. 2.3(f)). This alternative is applicable to lateral loads only, and is commonly referred to as the "shear beam" method.

2.2.4 Method of Solution

The solution of the equilibrium equations is performed by means of the tri-diagonal method, described in detail in the following chapter. This method calls for the division of the structure into sub-structures called units. In the present analysis, each floor is treated as a separate unit, with the solution starting from the topmost floor and proceeding to the lower floors. The solution process yields the displacements of the joints of the structure.

2.2.5 Member Stress Resultants

After all the joint displacements have been evaluated, each member is considered individually and the member end displacements are determined from the joint displacements. The member end displacements are then used to compute the member end forces. In the case of slabs, the maximum positive moments along the girder spans of the loaded slabs are also determined by superimposing the appropriate bending moment diagrams. For unloaded slabs, the midspan girder moments are computed instead. The member forces thus evaluated will henceforth be collectively referred to as the member stress resultants.

2.3 Loads

The structure may be analyzed for several loading conditions simultaneously. The analysis algorithm is equipped to handle the following four types of loads, which are commonly applied in building analysis:

1. Superimposed uniformly distributed dead loads acting on the panels.

2. Uniformly distributed live loads acting on the panels.

3. Arbitrary uniformly distributed loads acting on the panels.

4. Arbitrary joint loads acting at the joints of the structure, e.g., wind loads, earthquake loads, etc.

When the structure is being analyzed for dead loads, the self weight of the structure is considered in addition to the prescribed superimposed dead loads, and the structure is analyzed for the resulting total dead loads. The self weights of the members of the structure are computed internally, the self weights of the slabs being computed from the slab depths, and the self weights of the columns from the column concrete dimensions.

Two options are provided for live load analysis. The structure may either be analyzed with the full live load on all the panels acting simultaneously, or in such a manner that the maximum possible positive and negative values of each individual stress resultant are determined. The procedure employed to evaluate the maximum possible stress resultants is described in detail in the next chapter. However, the essence of the procedure is as follows:

1. In addition to the specified loading conditions, the structure is also analyzed for the specified uniformly distributed live

load acting on each panel separately. Hence, the number of loading conditions in addition to the specified loading conditions is equal to the total number of panels.

2. The maximum positive and negative live load stress resultants of each member are computed incrementally by evaluating the stress resultants of the member due to the live loading of each panel separately. The sign of a member stress resultant due to the loading of any particular panel determines whether it should be added into the corresponding maximum positive or maximum negative stress resultant of the member. Proceeding in this manner, the maximum positive and negative live load stress resultants of each member are accumulated by selectively summing the stress resultants obtained by loading each panel individually.

2.4 Loading Combinations

Since one or more loadings may act simultaneously upon a structure, it is necessary to be able to combine the effects of different loadings and compute the resultant member forces. The facility for combining different loadings is included in the model and several loading combinations may be specified. The maximum possible values of the member forces due to the specified loading combinations are the forces for which the members must be designed, and will be referred to as the member design quantities.

A loading may be specified to be reversible, meaning that it may act in both directions. This provision is useful in reducing the number of loadings and loading combinations that have to be specified. Since a reversible loading may act in both directions, the member stress resultants due to it may change signs. This fact is taken into account by taking the absolute values of the member stress resultants due to the reversible loading and adding them with the appropriate signs to both the maximum positive and maximum negative stress resultants of the member. The facility for specifying reversible loadings does not extend to dead loads and live loads, which are regarded as being non-reversible by nature.

Two types of loading combinations are incorporated in the model. These are:

a. "AND" type combinations, which perform the summation of different loadings to yield the maximum possible values.

b. "OR" type combinations, which provide the capability for comparing the stress resultants of two loadings and adding the larger of the two. The two loadings compared may be two non-reversible loadings, a reversible and a non-reversible loading, or two reversible loadings.

Along with each loading that is part of a loading combination, a factor by which the stress resultants of that loading should be multiplied may also be specified. Thus, for instance, an acceptable loading combination is

$$1{\cdot}1\ DL+(0.9LL\ OR\ 1.0W)$$

where DL represents the dead loads, LL the live loads and W the wind loads acting on the structure.

2.5 Design Quantities

The design quantities, defined above as the maximum possible member forces due to the specified loading combinations, are described below.

2.5.1 Slabs

The design quantities for a slab consist of the maximum positive and negative end moments and the maximum positive and negative moments along the span of the four girders bordering the slab, as shown in Fig. 2.4. The design quantities are obtained by selectively summing the stress resultants of the different loadings in accordance with the loading combination data.

When a space frame analysis is performed, the stress resultants of all the four girders bordering a slab are evaluated simultaneously, and the loading combination procedure yields all the design quantities of the slab. When a plane frame analysis is performed, on the other hand, the loading combination procedure yields only the design quantities of the girders in the plane frame analyzed. Hence, the design quantities of a slab can only be completely determined after the four plane frames containing the girders bordering the slab have been analyzed.

2.5.2 Columns

The loading combination procedure yields three sets of design quantities for each end of a column. These are:

1. The maximum axial load P and the corresponding moments about the X and Y axes, M_x and M_y, as shown in Fig. 2.5.
2. Maximum M_x and the corresponding P and M_y.
3. Maximum M_y and the corresponding P and M_x.

Hence, the design quantities for each column consist of six sets of values of P, M_x and M_y.

When a plane frame analysis is performed, the combination procedure yields an axial load and one moment per set. As such, the design quantities of a column can only be completely determined after the two plane frames containing the column have been analyzed. Moreover, since the two plane frame analyses yield two different axial loads, the larger of the two is arbitrarily taken as the design quantity.

2.6 Design

The slab and column design procedures incorporated into the model are based upon the recommendations of the ACI Code. In addition, the model attempts to provide capabilities which allow a designer to use his engineering judgment and experience in order to arrive at a good design.

2.6.1 Member Groups

For aesthetic or other architectural requirements, or with a view towards economy in construction, it is often desirable to make groups of members identical or similar in some respect. The facility for specifying such member groups has been incorporated into the model. For instance, the designer may stipulate that the depth of all the corner slabs on a particular floor level shall be the same, or that the column dimensions shall only be stepped every two stories, etc.

Although the ability to specify member groups will not in general lead to the design of structures of minimum weight, it will normally assist significantly towards the design of economical structures, which are at the same time acceptable to both the architect and the contractor.

2.6.2 Types of Member Groups

Two distinct categories of member groups are included in the model:

1. Preassigned groups - these are member groups specified as part of the input data.

2. Program selected groups - these are internally selected groups of members which satisfy specified constraints.

In the case of slabs, only preassigned groups are allowed. Within the category of preassigned groups, the following three types of slab groups are acceptable:

i. Groups of slabs of fixed depth.

ii. Groups of slabs of equal depth.

iii. Groups of slabs that are identical with respect to slab depth and steel provisions.

In the case of columns, both preassigned groups and program-selected groups are provided for, and within each of these categories two types of groups can be specified. These are:

i. Groups of columns of the same dimensions.

ii. Groups of identical columns, i.e., columns having the same dimensions and longitudinal reinforcement.

2.6.3 Design on Basis of Partial Analysis of Structure

In practice, if a designer requires only a crude preliminary design for a structure or if the structure is repetitive, having similar interior slabs etc., the designer may elect to design only the typical components of the structure and derive the design of the remaining components from these.

Consider, for example, the typical floor shown in Fig. 2.6. It may conceivably be adequate to design only one corner slab, two side slabs and one interior slab, and then make the remaining slabs identical to the designed slabs. In order to obtain design quantities for the four "design slabs", it is clearly sufficient to analyze only the six plane frames indicated by heavy lines in Fig. 2.6, instead of analyzing all the plane frames or the space frame. Going a step further, it may even be considered adequate to analyze only the four plane frames marked A in the figure. In that case, however, the plane frame analyses do not yield all the design quantities needed for the design slabs. To remedy this situation, fictitious quantities are created within the model in lieu of the missing design quantities, based upon the available ones.

The criterion used for choosing the control slab from a group of slabs is the number of sides for which the plane frames analyzed provide design quantities. In other words, that slab from each group for which the maximum percentage of design quantities are available is chosen as the control slab. An analogous criterion is used to select the "control column" from a group of columns.

2.6.4 Design of Members

The model is equipped to design the members either by the working stress method or by ultimate strength design procedures. The design procedures for slabs and columns are described below.

2.6.4.1 Slab Design Procedure

The depths of the slabs are determined prior to the analysis of the structure, in accordance with the empirical values suggested in Section 2104(d) of the ACI Code.

The design quantities of a slab are then used to determine the amounts of compressive and tensile steel required for the slab. Before this is done, however, the design quantities of the slab are converted into column and middle strip moments in accordance with Table 2103(c) of the ACI Code. The positive and negative design quantities at the twelve locations indicated in Fig. 2.7(a) yield positive and negative strip moments at the eighteen locations indicated in Fig. 2.7(b). Finally, the areas of steel required for the longitudinal and lateral column and middle strips of the slab are determined from the strip moments by means of standard working stress or ultimate strength design procedures. The implementation of these procedures is described in detail in the next chapter.

If a slab has been specified to be part of a group of identical slabs, this fact is taken into account while determining the steel required for the slab.

2.6.4.2 Column Design Procedure

The design of a column is performed in the following manner. First, the effective length and the corresponding strength reduction factor for the column are computed in accordance with the provisions of Sections 915 and 916 of the ACI Code, and the column design quantities are increased to compensate for the strength reduction, if any. Next, the increased design quantities are used to check the assumed section by means of the working stress or the ultimate strength design method. An iteration procedure, described in the following chapter, is used to arrive at the best possible design for the column.

If a column is part of any group of columns, this fact is recognized and taken into account in the design.

2.7 Checking of Structure

As mentioned in Chapter 1, the ability to check a previously designed structure is included in the capability of the model. The code adopted for the checking procedure is the current ACI Code.

The following data pertaining to the structure is provided to the checker by the designer:

1. The geometry of the structure.

2. The slab depths and the main steel reinforcements of the slabs.

3. The concrete and longitudinal steel areas of the columns.

The remaining data required for the analysis of the structure must be supplied by the checker, and includes:

1. Analysis alternative.

2. Loading data.

3. Loading combinations.

4. Constants and criteria, e.g., allowable stresses, etc.

It is to be noted that the above data are provided independently by the checker, and may be entirely different from the corresponding data used by the designer.

The structure is analyzed using the above data, and design quantities are obtained for the members of the structure. Whereas the member design quantities are used to design the members when the model is applied to design, the design quantities are used only to check the adequacy of the members when the model is applied to checking.

The important difference between checking and design is that there are no member groups to be considered when checking a structure. Otherwise, the checking procedures for the members resemble the design procedures closely. The checking of the slabs and columns is done as follows:

a) Slabs. If the structure is being checked by the working stress method, the steel and concrete stresses at all the critical points of the slab are determined, and a message is output whenever the calculated stresses exceed the allowable stresses. If the ultimate strength design method is used, the values of the resisting moment of the slab at all the critical points are computed, and a message is output whenever the resisting moment is less than the acting ultimate moment.

b) Columns. The safety of a column is checked by means of the ACI Code formulas for working stress or ultimate strength design, as required by the checker. When a column is found to be unsafe, an appropriate message is output.

2.8 Quantity Take-Off

After the design of a structure is complete, the designer may request a quantity take-off of the designed structure.

The quantity take-off procedure computes the total volume of concrete and weight of steel required. The weight of steel includes only the bending reinforcement of the slabs and the longitudinal reinforcement of the columns. The shear reinforcement of the slabs and the lateral ties for the longitudinal reinforcement of the columns are not taken into account.

CHAPTER 3

IMPLEMENTATION OF MODEL

This chapter describes the procedures implemented in the model described in Chapter 2. The topological relations required during the processing of the structure are described first. Next, the details of the analysis algorithm are given, followed by a description of the member design procedures. Finally, a description of the checking and quantity take-off procedures is presented.

3.1 Topological Relations

The topological relations of the structure used during processing are listed below. As described in Art. 2.1, the present model deals only with regular structures, and thus all the required relations can be obtained internally by simple calculations. The numbering schemes used for space and plane frames are illustrated in Fig. A.1, and the necessary calculations are given in Appendix A.

For structures with an irregular layout, containing setbacks, discontinuous column or bay lines, missing columns or slabs, etc., the equivalent topological information would have to be supplied as input data.

3.1.1 Space Frames

The information required for a given slab of the structure is:

1. The bay and aisle numbers of the slab.
2. The corner joint numbers of the slab.
3. The member numbers of the girders along the edges of the slab.

4. The numbers of the four slabs adjacent to the slab.

5. The numbers of the four frames bordering the slab.

The information required for a given joint of the structure is:

1. The longitudinal and lateral frame numbers intersecting at the joint.

2. The member numbers of the members framing into the joint.

3. The numbers of the joints at the far ends of members framing into the joint.

4. The numbers of the four slabs adjacent to the joint.

3.1.2 Plane Frames

As shown in Fig. A.1, the same members are assigned different numbers internally when they are part of space and plane frames. Some of the relations required for plane frames require the corresponding space frame numbers of members. The information required for a given girder of a plane frame is:

1. The corresponding space frame girder number.

2. The numbers of the two slabs adjacent to the girder.

The information required for a given joint of a plane frame is:

1. The space frame member numbers of the members framing into the joint.

2. The numbers of the four slabs adjacent to the joint.

Most of the relations listed above are required many times during the processing of a structure. Because they can be generated very quickly, however, the required information is recreated each time.

3.2 Analysis

3.2.1 General Comments

The analysis of the structure is performed by the stiffness method. The tri-diagonal procedure[3] is employed for the solution of the equilibrium equations, treating each story as a separate substructure or unit. The stiffness matrix of the structure is assembled story by story, and is of the form:

$$\left[\begin{array}{c:c:c:c:c:c}
K_{11} & K_{12} & 0 & 0 & & 0 \\ \hdashline
K_{21} & K_{22} & K_{23} & 0 & & 0 \\ \hdashline
0 & K_{32} & K_{33} & K_{34} & & 0 \\ \hdashline
 & & & & \ddots & K_{n-1,n} \\ \hdashline
0 & 0 & & & K_{n,n-1} & K_{nn}
\end{array}\right] \tag{3.1}$$

in which the K_{ii} represent the direct stiffnesses of the substructures, referred to herein as the floor stiffnesses, $K_{i,i-1}$ and $K_{i,i+1}$ represent the cross-stiffnesses between adjacent floors, and n is number of stories.

The computer time required to obtain a complete solution by the tri-diagonal procedure may sometimes prove to be excessive, because the method relies heavily upon the intermediate use of scratch storage. This situation could be alleviated at times by increasing or reducing the number of substructures within the primary memory capacity of the computer. The "overhead" time required for the use of scratch storage increases linearly with the number of substructures, while the solution time for each substructure increases in proportion to the cube of the number of unknowns per substructure. Hence, the optimal subdivision may not always be the one chosen for this study. However, this disadvantage is offset by the advantage of being able to maintain a uniform bookkeeping procedure throughout.

3.2.2 Stiffness Matrices of Members

The stiffness matrix of the structure is assembled from the stiffness matrices of the members expressed with respect to the global coordinate axes X, Y, Z, shown in Fig. 2.1. As is conventional in matrix structural analysis, the stiffness matrices of the members are first expressed with respect to their local member axes, and then transformed into the global axes of the structure. The derivation of the global member stiffness matrices is described below. In the program, the derived global member matrices are used directly as described in Art. 3.2.3, because it would prove too time-consuming to evaluate formally the global matrix for each member.

3.2.2.1 Space Frame Members

The stiffness matrix of a space frame member with reference to its local member axes x, y, z, shown in Fig. 3.1, is:

$$K_m^* = \left[\begin{array}{c|c} HK_{BB}^* H^t & -HK_{BB}^* \\ \hline -K_{BB}^* H^t & K_{BB}^* \end{array}\right] = \left[\begin{array}{c|c} K_{AA}^* & K_{AB}^* \\ \hline K_{BA}^* & K_{BB}^* \end{array}\right] \tag{3.2}$$

in which K_{BB}^* is the stiffness matrix of end B of the member, and H is the translation matrix from end B to end A. The matrix K_{BB}^* is given by:

$$K^*_{BB} = \begin{bmatrix} AE/L & 0 & 0 & 0 & 0 & 0 \\ 0 & 12EI_z/L^3 & 0 & 0 & 0 & -6EI_z/L^2 \\ 0 & 0 & 12EI_y/L^3 & 0 & 6EI_y/L^2 & 0 \\ 0 & 0 & 0 & GJ/L & 0 & 0 \\ 0 & 0 & 6EI_y/L^2 & 0 & 4EI_y/L & 0 \\ 0 & -6EI_z/L^2 & 0 & 0 & 0 & 4EI_z/L \end{bmatrix} \tag{3.3}$$

The elastic properties of the members are evaluated considering the concrete cross-section only. It should be noted that the shearing distortion terms due to L/GA_y and L/GA_z are ignored. For the types of members under consideration, this simplification is deemed to be justifiable. The translation matrix H is given by:

$$H = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & -L & 0 & 1 & 0 \\ 0 & L & 0 & 0 & 0 & 1 \end{bmatrix} \tag{3.4}$$

When the member axes do not coincide with the global axes of the structure, the member stiffness submatrices with respect to the global axes are obtained by applying the rotational transformation:

$$K_{rs} = TK^*_{rs}T^t \tag{3.5}$$

where T is the rotation matrix from member to global coordinates. Hence, the member stiffness matrix with respect to the global axes, K_m, is obtained as:

$$K_m = \left[\begin{array}{c|c} THK^*_{BB}H^tT^t & -THK^*_{BB}T^t \\ \hline -TK^*_{BB}H^tT^t & TK^*_{BB}T^t \end{array}\right] = \left[\begin{array}{c|c} K_{AA} & K_{AB} \\ \hline K_{BA} & K_{BB} \end{array}\right] \tag{3.6}$$

The rotation matrices corresponding to the three distinct space frame members are as follows:

a) Longitudinal Girders. For longitudinal girders the member axes coincide with the global axes X, Y, Z, so that T = [I], where I represents a unit (identity) matrix.

b) Lateral Girders. The relation between the member axes of a lateral girder and the global axes is shown in Fig. 3.2. The corresponding rotation matrix is given by:

$$T = \begin{bmatrix} 0 & -1 & 0 & 0 & 0 & 0 \\ 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & -1 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{3.7}$$

c) Columns. The relation between the member axes of a column and the global axes is shown in Fig. 3.3. The corresponding rotation matrix is given by:

$$T = \begin{bmatrix} 0 & 0 & -1 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & -1 \\ 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \end{bmatrix} \tag{3.8}$$

3.2.2.2 Plane Frame Members

There are three generalized displacements at each end of a plane frame member as shown in Fig. 3.4, and the stiffness matrix corresponding to end B of the member, with reference to the local member axes is:

$$K_{BB}^{*} = \begin{bmatrix} AE/L & 0 & 0 \\ 0 & 12EI_y/L^3 & 6EI_y/L^2 \\ 0 & 6EI_y/L^2 & 4EI_y/L \end{bmatrix} \qquad (3.9)$$

The translation matrix from end B to end A of the member is:

$$H = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & -L & 1 \end{bmatrix} \qquad (3.10)$$

The stiffness matrices of the longitudinal and lateral plane frame members with respect to the global axes are obtained as described below.

a) Longitudinal Frames. The rotation matrices corresponding to the girders and columns of a longitudinal frame are as follows:

i) Girders. The member axes of a longitudinal plane frame girder coincide with the global axes, so that $T = [I]$.

ii) Columns. The rotation matrix of a column is:

$$T = \begin{bmatrix} 0 & -1 & 0 \\ 1 & 0 & 0 \\ 0 & 0 & 1 \end{bmatrix} \qquad (3.11)$$

b) Lateral Frames. Lateral frames are treated as longitudinal frames for the purpose of analysis. There is no complication in doing so, because only an imaginary rotation of the lateral frames into the plane of the longitudinal frames is involved. The stiffness matrices corresponding to longitudinal plane frame girders and columns apply without change to lateral frames, except that in the stiffness matrix of the columns the moment of inertia about the member z axis, I_z, must be used in place of the moment of inertia about the member y axis, I_y.

3.2.3 Mapping from Member to Joint Submatrices

The stiffness matrix of a space frame member (Eq. (3.6)) consists of 144 elements. In the conventional procedure, all 144 elements, or at least the 36 elements of K_{BB}, are stored in the computer memory and the submatrices K_{AA}, K_{AB}, K_{BA} and K_{BB} are copied or "mapped" into appropriate locations in the stiffness matrix of the structure. A considerably more efficient procedure was developed and incorporated into the present model. The procedure is based upon the observation that there are only eight variables in all the member stiffness matrices, namely:

Variable No.	1	2	3	4	5	6	7	8
Variable	$\frac{AE}{L}$	$\frac{EI_y}{L}$	$\frac{EI_y}{L^2}$	$\frac{EI_y}{L^3}$	$\frac{EI_z}{L}$	$\frac{EI_z}{L^2}$	$\frac{EI_z}{L^3}$	$\frac{GJ}{L}$

It is further noted that, with the particular rotation matrices needed in this study, each of the 6 x 6 submatrices contains only ten non-zero terms, as in Eq. (3.3). Given the values of the above eight variables, the appropriate constant multipliers (2,4,6,12) and signs of the non-zero terms, as well as the location of the non-zero terms in the stiffness matrix, it is an easy matter to assemble the member stiffness matrix when needed. Indeed, it is possible to use the same information to determine the contribution of the member to the stiffness matrix of the structure without ever physically assembling the member stiffness matrix, thus resulting in a considerable saving of both computer memory space and processing time.

The implementation scheme developed requires the establishment of three lists for the description of the global stiffness matrix for each type of member. The lists describe the non-zero terms in the submatrices K_{AA}, K_{AB}, K_{BA} and K_{BB}. The required lists for space frame members are given in Table 1. The interpretations of the lists are as follows:

a) Source List. The source list points to the variables that correspond to the non-zero terms in each of the submatrices K_{AA}, K_{AB}, K_{BA} and K_{BB}. Referring to Table 1, the second element in the source list for space frame longitudinal girders is 7, because the second non-zero term in the stiffness submatrices (proceeding by rows) is $12EI_z/L^3$, which contains the 7th variable EI_z/L^3.

It is to be noted that the variables always occur in the same sequence in all four submatrices, so that a single source list serves for all submatrices.

b) Sign and Multiplier List. The sign and multiplier list is a two-dimensional list which defines the signs and multipliers of the non-zero elements in each of the four submatrices K_{AA}, K_{AB}, K_{BA} and K_{BB}. Referring again to Table 1, the boxed numbers in the sign and multiplier list for space frame longitudinal girders indicate that the sign and multiplier of the second non-zero term in the stiffness submatrices (proceeding by rows) is 12 for submatrices K_{AA} and K_{BB}, and -12 for submatrices K_{AB} and K_{BA}.

c) Location List. The location list defines the location of the non-zero elements in the submatrices K_{AA}, K_{AB}, K_{BA} and K_{BB}. It is the same for all four submatrices and consists of the row and column numbers corresponding to the non-zero elements in the submatrices.

In Table 1, the circled elements in the location list for space frame longitudinal girders indicate that the second non-zero term in the submatrices K_{AA}, K_{AB}, K_{BA}, and K_{BB} is located in row 2, column 2.

The source, sign and multiplier, and location lists required for plane frame members are given in Table 2. The plane frame member lists are five elements in length, because the global stiffness submatrices of plane frame girders and columns contain five non-zero terms in all cases.

In order to evaluate the contribution of the stiffness submatrices of a member to the stiffness matrix of a structure, the lists described above are used as follows. First, the values of the eight variables AE/L . . . GJ/L are computed from the elastic properites of the given member. Next, the source and sign and multiplier lists for the member are used to evaluate the non-zero terms in the member stiffness submatrices. The non-zero terms are then copied directly into the appropriate joint submatrices in the locations defined by the location list for the member.

3.2.4 Generation of Joint Submatrices

The solution procedure requires only the current floor stiffness matrix K_{ii} and the cross-stiffness matrix $K_{i,i-1}$ to be in memory concurrently. The matrices K_{ii} and $K_{i,i-1}$ are composed of joint submatrices associated with the joints in floors i and i-1. The generation of the joint submatrices required to assemble K_{ii} and $K_{i,i-1}$ is described below.

Each joint within floor i is considered in turn. The stiffness matrices associated with joint j consist of a submatrix representing the stiffness of the joint itself, k'_{jj}, and submatrices,

k'_{jg}, representing the cross-stiffnesses between the joint and each of its neighboring joints, g. The stiffness submatrix of the joint, k'_{jj}, and the cross-stiffness submatrices corresponding to the neighboring joints on the same floor belong to the floor matrix K_{ii}, whereas the cross-stiffness submatrix between the joint and the neighboring joint in the adjacent upper floor belongs to $K_{i,i-1}$. The cross-stiffness matrix $K_{i,i-1}$ is a diagonal matrix of the form:

$$K_{i,i-1} = \begin{bmatrix} k_1 & & & \\ & k_2 & & 0 \\ & & \ddots & \\ 0 & & & k_r \end{bmatrix}$$

where r is the number of joints per floor and $k_1, \ldots k_r$ are the cross-stiffness submatrices.

In the program, only the upper triangular submatrices of K_{ii} are explicitly determined, because the lower triangular submatrices are easily obtained by symmetry. The upper triangular submatrices of K_{ii} and the submatrices of $K_{i,i-1}$ associated with joint j are generated in the following manner. The member numbers of the members framing into the joint are determined, and taking each member in turn, its contribution to the joint submatrices is determined. The contribution of a member depends upon its orientation, and is different for girders and columns. The orientation of the members meeting at a joint is shown in Fig. A.2(b) for space frames and in Fig. A.3(a) for plane frames. The different cases that occur and the corresponding actions are as follows:

a) Girders with j as their positive end. The stiffness submatrices K_{AA} and K_{AB} of the girder are added into k'_{jj} and k'_{jg}, respectively, where g is the joint number of the negative end of the girder.

b) <u>Girders with j as their negative end</u>. The stiffness submatrix K_{BB} of the girder is added into k'_{jj}.

c) <u>Upper Columns</u>. The stiffness submatrices K_{AA} and K_{AB} of the column are added into k'_{jj} in K_{ii} and k_j in $K_{i,i-1}$, respectively.

d) <u>Lower Columns</u>. The stiffness submatrix K_{BB} of the column is added into k'_{jj}.

The lists described in Art. 3.2.3 for the member stiffness matrices correspond to the complete stiffness matrices of the members. Consequently, the joint submatrices evaluated using these lists represent the joint submatrices of a structure in which all degrees of freedom are considered. Hence, the evaluated joint submatrices correspond to Alternative 1 for both space and plane frame analyses.

3.2.5 <u>Mapping from Joint to Floor Matrices</u>

The joint submatrices evaluated for Alternative 1 are mapped into the floor matrices, K_{ii} and $K_{i,i-1}$, in accordance with the alternative chosen for analysis. Consider, for instance, the typical plane frame joint submatrix:

$$\begin{bmatrix} k_{11} & k_{12} & k_{13} \\ k_{21} & k_{22} & k_{23} \\ k_{31} & k_{32} & k_{33} \end{bmatrix}$$

If this represents the joint submatrix k'_{22} for joint 2 in floor 1 of the plane frame shown in Fig. 3.5, it is copied into rows and columns 4, 5, and 6 of the floor matrix, K_{11}, if the structure is being analyzed by Alternative 1. If, on the other hand, Alternative 2 has been chosen for analysis, row 2 and column 2 of the joint submatrix are omitted, because these correspond to column axial deformations, which are neglected in

Alternative 2. The remaining rows and columns 1 & 3 of the joint submatrix are then copied into rows and columns 3 & 4 of the floor matrix, K_{11} (Fig. 3.6). The cross stiffness joint submatrices, k'_{jg}, are mapped into the floor matrices in an identical manner.

The procedure implemented for mapping the joint submatrices into the floor matrices for the various alternatives of analysis is as follows. A source array containing the row and column numbers of the elements to be extracted from a joint submatrix is generated by bookkeeping. For a particular alternative, the source array is made up of constants and is the same for all joints in the structure. It is, therefore, generated only once. A second array, called the destination array, containing the row and column numbers of the locations in the floor matrices into which the elements of the joint submatrix are to be placed is also generated by bookkeeping. However, the elements of the destination array are variables which depend upon the joint number concerned and the analysis alternative chosen. Consequently, the destination array has to be recreated for each joint submatrix.

The source and destination arrays for the various alternatives are described in the next two articles. For ease of explanation, the plane frame alternatives are described before the space frame alternatives.

3.2.5.1 Plane Frame Alternatives

The mapping for the various alternatives falls into two categories - those of omission and superposition. In the case of plane frames, Alternatives 2, 5 and 6 fall into the category of omission and Alternatives 3 and 4 fall into the category of superposition.

The source and destination arrays for all plane frame alternatives are given in Table 3. As an explanation, the derivation of these arrays for Alternative 2 is described below. In addition, pertinent explanations relating to the other alternatives are also given.

<u>ALTERNATIVE 2</u> (Neglect column axial deformations):

This is an example of omission. In order to neglect the column axial deformations, it is necessary to omit the second row and column from every joint stiffness matrix before copying it into the matrix of the structure.

a) <u>Source Array</u>. Only four elements from the joint submatrix have to be copied into the structure matrix. These are the elements in the first and third rows and columns, and lead to the following source array:

$$\begin{bmatrix} 1,1 & 1,3 \\ 3,1 & 3,3 \end{bmatrix}$$

b) <u>Destination Array</u>. There are two degrees of freedom per joint of the structure. If J is the number of the joint concerned, then for the diagonal submatrices in K_{ii} and $K_{i,i-1}$ corresponding to the joint, the base addresses, JJ and JK, in terms of which the destination locations are calculated, are given by:

$$JJ = (J-1) \times 2$$

$$JK = JJ$$

For the off-diagonal submatrices in K_{ii}:

$$JJ = (J-1) \times 2$$

$$JK = (JN-1) \times 2$$

where JN represents the adjacent joint numbers. Since only the upper triangular submatrices of K_{ii} have to be generated, and the numbering of the joints in a floor is sequential as shown in Fig. A.1, the adjacent joint number of interest is J+1. Hence, for the off-diagonal submatrices:

$$JK = J \times 2$$

The required destination array for all the joint submatrices is given by:

$$\begin{bmatrix} JJ+1, JK+1 & JJ+1, JK+2 \\ JJ+2, JK+1 & JJ+2, JK+2 \end{bmatrix}$$

<u>ALTERNATIVE 3</u> (Neglect girder axial deformations):

When the girder axial deformations are neglected, the relative lateral deformation between girder ends is equal to zero so that the lateral translation or sidesway of all the joints on a floor level is the same. Hence, the number of degrees of freedom of a floor is reduced by (r-1), where r is the number of joints in the floor. This is an example of combination, because the lateral stiffness of the frame is now equal to the sum of the lateral stiffnesses of the columns across the frame. Since the girder axial deformations are neglected, it is necessary to set the AE/L terms in the member stiffness matrices of the girders equal to zero before mapping them into the joint submatrices.

In all the alternatives that involve combination, the degrees of freedom that are common to more than one joint are numbered after all the distinct degrees of freedom at each joint. The joint stiffness submatrices assembled as if for Alternative 1 can then be mapped into the story stiffness matrices in accordance with the appropriate source and destination arrays.

ALTERNATIVE 4 (Neglect girder and column axial deformations)

This alternative is a combination of Alternatives 2 and 3, and is an example of omission plus combination.

ALTERNATIVE 5 (No sidesway)

This is an example of omission. Only the bending rotations are considered, so that only element k_{33} of the joint submatrices is copied into the structure matrix.

ALTERNATIVE 6 (Sidesway only)

This is an example of omission plus combination. There is only one deformation per story, and hence no off-diagonal submatrices in the floor matrix K_{ii}. Since the axial deformations of girders are neglected, the AE/L terms must be set to zero in the stiffness matrices of the girders before copying them into the joint stiffness matrices.

3.2.5.2 Space Frame Alternatives

Among the space frame alternatives, Alternative 1 allows all six deformations at a joint whereas Alternatives 2, 3 and 10 are examples of omission and the remaining alternatives involve omission plus combination. Alternatives 1, 2, 3, 4, 7 and 10 entail very straightforward mapping and require no supplementary explanations. Their source and destination arrays are given in Table 4.

In the case of Alternatives 5, 6, 8 and 9, the mapping itself is straightforward but the joint submatrices must first be modified before being copied into the structure matrix. This modification, which is necessitated by the fact that the floors are assumed to act as rigid bodies, is described below.

3.2.5.3 Modification of Joint Submatrices for Rigid Floors

ALTERNATIVES 6 and 9:

When the axial and transverse deformation of a floor slab are neglected, the in-plane floor displacements are reduced from $3 \cdot r$ to 3, namely translations in the X and Y directions and a rotation about the vertical axis as shown in Fig. 2.2(f). The treatment of the member stiffness matrices and the joint submatrices to handle the change in the generalized floor displacements is explained below.

a) Modification of Member Submatrices.

i) Girders. Since there is no relative axial deformation, no relative transverse deformation and no relative rotation between the girder ends, it is necessary to set all the AE and EI_z terms in the member stiffness matrix equal to zero before copying into the joint stiffness matrix. No other modification is required for the girder submatrices.

ii) Columns. Since all the relative deformations between the column ends still exist, the complete column member stiffness submatrices must be copied into the joint submatrices without change.

b) Modification of Joint Submatrices.

The joint displacements, actions at joints, and column stiffnesses associated with the rigid body motions of the floors must be transformed to a common set of reference axes for each floor. Since the two ends of a girder lie on the same floor, no transformation of girder stiffnesses is required. Hence, the off-diagonal joint submatrices in K_{ii}, which correspond to girder stiffnesses alone require no modification.

Referring to Fig. 3.7, taking the geometric center of the floor slab, 0, as the origin of coordinates, the displacements of any joint j associated with the rigid body motion of the floor in terms of the motion of point 0 are:

$$\delta_{j1} = \delta_{01} - y_j\ \delta_{06} \tag{3.12}$$

$$\delta_{j2} = \delta_{02} + x_j\ \delta_{06} \tag{3.13}$$

and $$\delta_{j6} = \delta_{06} \tag{3.14}$$

where δ_{j1}, δ_{j2} and δ_{j6} represent the x translation, y translation and z rotation of joint j, respectively, and δ_{01}, δ_{02} and δ_{06} denote the corresponding displacements for point 0. The terms x_j and y_j are the x and y components of the radial distance from point 0 to joint j. The relations between the displacements of joint j and point 0 can be expressed in matrix form as

$$\delta_j = T_j^t\ \delta_0 \tag{3.15}$$

where

$$T_j = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & 0 \\ -y_j & x_j & 0 & 0 & 0 & 1 \end{bmatrix} \tag{3.16}$$

The transformation matrix T_j^t represents a geometric operator which transforms displacements of the floor at point 0 into displacements at joint j.

By the principle of contragradience the relation between the actions at point 0 in terms of the actions at any joint j are

$$P_0 = T_j P_j \tag{3.17}$$

in which P_0 and P_j represent the actions at point 0 and joint j respectively.

The desired modification of the joint stiffness submatrices is obtained by first writing the action-displacement equation for a joint as

$$P_j = K_j \delta_j \tag{3.18}$$

Substituting Eq. (3.15) into Eq. (3.18) and then substituting the resulting equation into Eq. (3.17) yields

$$P_0 = T_j K_j T_j^t \delta_0 \tag{3.19}$$

Since Eq. (3.19) is the action-displacement equation relating floor actions to floor displacements, the required modified joint stiffness matrix, K_0, is given by the congruence transformation

$$K_0 = T_j K_j T_j^t \tag{3.20}$$

In the program the transformation of the joint submatrices is not performed by actual matrix multiplication, which would be extremely time consuming, but rather by multiplying only the appropriate terms by x_j or $-y_j$.

After the joint submatrices are modified in accordance with Eq. (3.20), they are mapped into the structure matrix using the source and destination arrays given in Table 4.

<u>ALTERNATIVES 5 and 8</u>:

When the transverse deformations of the floor slab are neglected, the generalized in-plane floor displacements are reduced from 3·r to as many translations as the total number of plane frames in the structure plus a single rotation about the vertical axis as shown in Fig. 2.2(e). The modification of the member and joint stiffness matrices to reflect the change in the generalized floor displacements is exactly the same as for Alternatives 6 and 9 with the following two important differences:

a) In the member stiffness matrices of the girders only the EI_z terms are set equal to zero. The AE terms must be retained, because the axial deformations of the slabs are not neglected.

b) Since the axial deformations of the slabs are not neglected, the girders are subject to relative axial displacements between their ends. Because of this, the girder stiffness matrices require the same geometric transformation as the column stiffness matrices. This requirement is automatically met in the case of the diagonal joint submatrices, which are transformed in accordance with Eq. (3.20) just as for Alternatives 6 and 9. In addition, the off-diagonal joint submatrices in K_{ii}, which do not require any transformation for Alternatives 6 and 9, must also be transformed in accordance with Eq. (3.20). The source and destination arrays for Alternatives 5 and 8 which are required for mapping the modified joint submatrices into the structure matrix are given in Table 4.

When all the joints in a floor have been processed and the joint submatrices associated with the joints have been mapped into the floor matrices as described above, the generation of the matrices K_{ii} and $K_{i,i-1}$ is complete. The floor matrices are now ready for use in the solution process.

3.2.6 Loads

The types of loads included in the model were described in Art. 2.3. The loads are applied either directly as joint loads, or as uniform loads acting on the slabs. These loads must be transformed into joint load vectors, P, which are compatible with the analysis alternative chosen.

The generation of the appropriate joint load vectors for different loading conditions and analysis alternatives is explained in the following articles.

3.2.6.1 Joint Loads Due to Uniform Loads

In order to evaluate the joint loads due to a uniformly distributed load on a slab, it is necessary to make an assumption regarding the transfer of load from the slabs to the supporting columns. For an elastic analysis of the structure, Section 2103(a) of the ACI Code recommends that the structure be analyzed as a series of plane frames, the longitudinal and lateral frames being bounded by the center line of the panel on either side of the center line of the columns. The total load acting on the portions of the panels included in a plane frame is assumed to be borne by the plane frame. For the present model, the ACI Code recommendation is used for the plane frame analysis of the structure. Hence, if a plane frame girder has panels of width ℓ_1 and ℓ_2 on either side as shown in Fig. 3.8, and the intensity of the uniform loads on the panels is w_1 and w_2 respectively, the intensity of the equivalent uniform load on the girder is equal to $0.5\ (w_1\ell_1+w_2\ell_2)$.

For the space frame analysis, the girders are similarly assumed to be loaded by the total load acting on an area bounded by the center lines of the panels on either side of the girders. However, this assumption is used only to evaluate the moments in the girders. For static equilibrium the joint loads in the vertical direction are computed assuming that each column supports the total load enclosed by the center lines of the panels adjacent to the column.

For the space frame analysis, three different loading distributions were tested. These are:

1. The standard two way slab load distribution. This distribution does not satisfy statics for flat plates and was, therefore, rejected.

2. The two way slab load distribution plus fixed-end torsional moments applied to satisfy statics.

3. The plane frame distribution described above.

Since distributions 2 and 3 were found to provide comparable results and distribution 3 was easier to apply, the latter was chosen for inclusion in the analysis algorithm.

3.2.6.2 Storage of Loads

Since the solution of the structure proceeds by stories, the joint loads of each story are required separately. It is, therefore, convenient to generate the joint loads by stories, and have only the joint loads of one story at a time in memory during the solution process. The joint loads of a story will henceforth be referred to as story loads.

The joint load vectors for all the input loading conditions are determined on the assumption that the analysis alternative is space frame Alternative 1. As each loading is read, the joint load vector for each story is created and immediately stored on secondary storage. The joint load vectors can then be read off from secondary storage when they are required in the analysis procedure and modified for the appropriate analysis alternative as described in the next article. The advantage of generating the joint load vectors for space frame Alternative 1 is that if any data pertaining to the method of analysis of the structure is modified, it is not necessary to read the loading data once again and determine the corresponding joint load vectors.

3.2.6.3 Modification of Joint Load Vectors

As stated above, the joint load vectors created from the input loading data correspond to space frame Alternative 1. Therefore, whenever any other alternative is specified, the joint load vectors have to be modified. The modifications involved for the various alternatives are similar to the mapping of the joint stiffness matrices into the floor matrices (Art. 3.2.5), and also fall into the two categories of omission and superposition.

The lists used for mapping the joint load vectors from space frame Alternative 1 to the other space frame alternatives and to the plane frame alternatives are given in Tables 5 and 6 respectively. The joint load vectors for longitudinal plane frames are made up from joint loads acting in the directions of generalized displacements 1, 3 and 5 of a space frame joint as depicted in Fig. 3.9(a), whereas the joint load vectors for lateral plane frames consist of joint loads acting in directions 2, 3 and 4 as shown in Fig. 3.9(b). Since lateral plane frames are analyzed by treating them as longitudinal plane frames, it is necessary to reverse the sign of the joint loads acting in direction 4 before copying them into the lateral frame joint load vectors.

3.2.6.4 Treatment of Live Loads

When the structure is analyzed with the full live load on all panels, the live loads are treated exactly like all the other loading conditions. In order to evaluate the maximum effect due to live loads, however, additional loading conditions must be generated internally. The corresponding joint load vectors need not be created at the time when the loadings are read in, but can be generated as needed during the solution procedure. The creation of the additional vectors is different for space and plane frame analysis, as explained below.

For space frame analysis, each panel of the structure is loaded separately and forms one loading condition. Hence, the additional number of loading conditions is equal to the total number of panels in the structure. The fixed-end reactions of the idealized girders are determined and copied into the joint load vector corresponding to the loaded panel. The remaining terms in the vector are set equal to zero. The joint load vectors are created directly for the particular analysis alternative specified.

For plane frame analysis, each girder is loaded separately. Hence, the number of additional loading conditions is equal to the number of girders in the plane frame. The loading of a girder consists of loading the panels on both sides of the girder. For exterior plane frames there is only a panel on one side that can be loaded. The fixed-end reactions of the girder under consideration are determined and copied into the joint load vector corresponding to the loaded girder. As for space frames, the joint load vectors are created directly for the plane frame analysis alternative specified.

3.2.6.5 Self Weight of the Structure

The joint loads due to the self weight of the structure are evaluated just before the solution, and added into the joint load vector corresponding to the superimposed dead loads. If no superimposed dead loads are specified, the joint loads due to self weight constitute the total joint loads due to dead loads. The locations into which the joint loads have to be added are determined directly for the particular analysis alternative specified.

The joint loads due to the self weights of the slabs are computed as described for uniform loads in Art. 3.2.6.1. The joint loads due to the self weights of the columns are evaluated only for the analysis alternatives in which the column axial deformations are considered. The self weight of a column provides joint loads equal to half the weight of the column acting downwards at each end of the column.

3.2.7 Solution Procedure

The solution proceeds story by story, starting with the topmost story. The equilibrium equations of the complete structure are of the form:

$$\begin{array}{r} \text{Story } 1 \\ 2 \\ \cdot \\ \cdot \\ n \end{array} \begin{bmatrix} K_{11} & K_{12} & 0 & & & 0 \\ K_{21} & K_{22} & K_{23} & 0 & & 0 \\ 0 & K_{32} & K_{33} & K_{34} & & 0 \\ & & & \ddots & \ddots & \\ & & & & K_{n,n-1} & K_{nn} \end{bmatrix} \begin{bmatrix} D_{11} & D_{12} & \cdots & D_{1\ell} \\ D_{21} & D_{22} & \cdots & D_{2\ell} \\ \vdots & \vdots & & \vdots \\ D_{n1} & D_{n2} & \cdots & D_{n\ell} \end{bmatrix} = \begin{bmatrix} P_{11} & P_{12} & \cdots & P_{1\ell} \\ P_{21} & P_{22} & \cdots & P_{2\ell} \\ \vdots & \vdots & & \vdots \\ P_{n1} & P_{n2} & \cdots & P_{n\ell} \end{bmatrix} \quad (3.21)$$

in which K_{ii}, $K_{i,i-1}$ and n are as defined in Art. 3.2.1, D_{ij} are the deformations of story i due to loading j, and ℓ is the total number of loading conditions.

The solution is a straightforward elimination algorithm, and proceeds as follows. For any loading, say the jth, the first two rows of Eq. (3.21) yield

$$K_{11}D_{1j}+K_{12}D_{2j} = P_{1j} \tag{3.22}$$

$$K_{21}D_{1j}+K_{22}D_{2j}+K_{23}D_{3j} = P_{2j} \tag{3.23}$$

Eliminating D_{1j} from Eq. (3.23), using Eq. (3.22), gives

$$D_{2j} = K_{22}^{*-1}[P_{2j}^{*}-K_{23}D_{3j}] \tag{3.24}$$

in which

$$K_{22}^{*} = K_{22}-K_{21}K_{11}^{-1}K_{12}$$

$$P_{2j}^{*} = P_{2j}-K_{21}K_{11}^{-1}P_{1j}$$

Eliminating the story displacements successively, yields in general

$$D_{ij} = K_{ii}^{*-1}[P_{ij}^{*}-K_{i,i+1}D_{i+1,j}] \tag{3.25}$$

in which

$$K_{ii}^{*} = K_{ii}-K_{i,i-1}(K_{i-1,i-1}^{*})^{-1}K_{i-1,i} \tag{3.26}$$

$$P_{ij}^{*} = P_{ij}-K_{i,i-1}(K_{i-1,i-1}^{*})^{-1}P_{i-1,j}^{*} \tag{3.27}$$

For each story, the inverse of the effective floor stiffness matrix K_{ii}^{*}, the effective load vectors P_{ij}^{*}, and the cross stiffness matrix $K_{i,i-1}$ are evaluated and written on secondary storage for use during the back-substitution phase, before proceeding to the next story. For the last story, the procedure yields

$$D_{nj} = K_{nn}^{*-1}P_{nj}^{*} \tag{3.28}$$

which is the solution for D_{nj}. Next, backsubstituting into Eq. (3.25) and proceeding story by story from story n to story 1, all the unknown story displacements can be computed. For each story, the displacement vectors corresponding to each of the ℓ loading conditions are evaluated and written on secondary storage before proceeding to the next story.

3.2.8 Back-Substitution

The objective of the back-substitution process is the determination of the member end forces and the derived forces along the length of the girders, collectively referred to as the member stress resultants. The member end forces are computed from the member end deformations, which in turn are determined from the joint displacements of the structure. Therefore, the generalized story displacements written on secondary storage at the end of the solution process must be brought into primary memory. The member forces are evaluated story by story starting with the topmost story. For columns, it is necessary for the displacements of two stories to be in core simultaneously. This is accomplished by reading in the displacements of the first two stories when determining the member forces of the topmost story, and after that reading in only the displacements of the next lower story each time.

3.2.8.1 Evaluation of Member Deformations

For most of the analysis alternatives, the generalized story displacements correspond directly to the joint displacements. For space frame Alternatives 5, 6, 8 and 9, however, the story displacements must be transformed back into joint displacements using Eqs. (3.12) to (3.14).

The member end deformations are evaluated from the joint displacements by means of a straightforward mapping procedure. The lists used are given in Table 7 for the space frame alternatives, and in Table 8 for the plane frame alternatives. The internal numbering of the member end deformations of slabs and space frame columns is illustrated in Fig. 3.10, while the numbering for plane frame girders and columns is shown in Fig. 3.11.

3.2.8.2 Member End Forces

The member end forces act in the directions of the generalized member end displacements, and are given with respect to the global axes by the relation

$$\{p\} = [K_m]\{\delta_m\} + \{f\} \tag{3.29}$$

in which the vector δ_m represents the member end deformations, and f represents the fixed-end reactions of the members.

A space frame member has six stress resultants at each end of the member, whereas a plane frame member has three at each end. The only end stress resultants required for the design of the structure, however, are the bending moments and reactions of the girders, and the bending moments and axial forces of the columns. The required quantities are evaluated using the appropriate formulas extracted from Eq. (3.29).

3.2.8.3 Column Axial Forces

When the axial deformations of the columns are considered, the column axial forces are automatically evaluated as part of the stress resultants. However, when the axial deformations of the columns are neglected, the stress resultants in the direction of the column axial deformations are zero, and it is necessary to evaluate the column axial forces by statics as follows.

For a particular loading condition, the axial force in a column is evaluated as the sum of three quantities. These are:

a) The gravity loads acting upon the floor area supported by the column.

b) The sum of the unbalanced moment reactions of the girders framing into the column. The sign of the reaction is positive if the column is at the negative end of a longitudinal girder, or at the positive end of a lateral girder.

c) The cumulative value of the vertical load from all the upper floors. This quantity is obtained by maintaining an array which contains the current value of the vertical load from above. The array is updated as each story is processed. When computing the column axial forces due to dead loads, the self weight of the columns is also added into the array.

When the structure is analyzed as a series of plane frames, the above procedure yields two values for the axial force, one corresponding to the longitudinal frame and the other to the lateral frame the column occurs in. The larger of the two values is used for the design of the column.

3.2.8.4 Maximum Positive Moment along Girder Spans

In addition to the member end forces evaluated from Eq. (3.29), it is necessary for design purposes to evaluate the maximum positive moment along the girders.

For a loaded girder AB, the moment at a distance x from end A is given by:

$$M(x) = \frac{wx}{2}(L-x)-M_A-\frac{(M_B-M_A)x}{L} \tag{3.30}$$

in which w is the intensity of the uniform load, L is the length of the girder, and M_A and M_B are the girder end moments. The position of maximum moment may be obtained, using Eq. (3.30), as

$$\bar{x} = \frac{L}{2} - \frac{\Delta R}{w} \tag{3.31}$$

where

$$\Delta R = \frac{M_B - M_A}{L} \tag{3.32}$$

Using Eqs. (3.31) and (3.32) in Eq. (3.30), the maximum positive moment is obtained as:

$$M(\bar{x}) = \frac{w\bar{x}(L-\bar{x})}{2} - M_A - \Delta R \cdot \bar{x} \tag{3.33}$$

Although the maximum positive moment generally does not occur at the middle of a girder, it will henceforth be referred to as the midspan moment.

For a girder that is not loaded, the bending moment at midspan is evaluated in lieu of the maximum positive moment. It is given by

$$M\left(\frac{L}{2}\right) = -\frac{M_A + M_B}{2} \tag{3.34}$$

3.2.8.5 Live Load Stress Resultants

When the full live load is applied on all the panels of the structure, the live load member stress resultants are evaluated as described in Arts. 3.2.8.2 to 3.2.8.4. When the maximum stress resultants due to live loads have to be determined, however, the maximum positive and negative end moments and the maximum positive and negative midspan moments are accumulated as follows:

a) End Moments. The maximum positive and negative end moments are obtained by selectively summing the end moments due to the live loading of each panel of the structure. For example, if the end moment due to the loading of a particular panel is negative, it is added into the corresponding value for the maximum negative moment.

b) Midspan Girder Moments. As each panel loading is considered, it is necessary to determine whether the loading causes a positive or negative moment increment at midspan of the girder under consideration. For this calculation, it is considered satisfactory to assume that the maximum positive moment occurs at midspan.

The moment increment is computed using Eq. (3.33) if the girder is loaded, and Eq. (3.34) if it is not loaded. If the moment increment is positive, the corresponding end moments M_A and M_B are added into the sums of the end moments corresponding to the maximum positive midspan moment due to live loads, M_P^A and M_P^B. Similarly, if the moment increment is negative, M_A and M_B are added into the sums of the end moments corresponding to the maximum negative midspan moment, M_N^A and M_N^B. After all panel loadings have been considered, the accumulated values of M_P^A and M_P^B are used in Eq. (3.31) to determine the position of the maximum positive moment, $\bar{x}$, and then Eq. (3.33) is used to determine the maximum positive moment. The accumulated values of M_N^A and M_N^B are substituted into Eq. (3.34) to yield the maximum negative midspan moment.

3.3 Loading Combinations

The objective of the loading combination procedure is to determine the design quantities of the members, described in Art. 2.5, from the member stress resultants previously calculated. The stress resultants for each loading condition are stored in arrays so that when the stress resultants due to all the loading conditions have been determined, they can be combined as specified in the loading combination data in order to yield the required design quantities.

The admissible types of loading combinations, i.e., 'AND' and 'OR' combinations and the capability for specifying reversible loadings have been described in Art. 2.4. Stated more explicitly, these facilities allow several "loading units" to be selectively summed in order to yield the maximum possible stress resultants. A loading unit is defined herein as one of the following:

1. A non-reversible loading.

2. A reversible loading, i.e., one that can occur in both directions.

3. An 'OR' combination between two non-reversible loadings.

4. An 'OR' combination between a non-reversible loading and a reversible loading.

5. An 'OR' combination between two reversible loadings.

Because of the nature of dead loads, it is assumed that dead loads can never be part of an 'OR' combination, since a legitimate comparison cannot be made. Moreover, both dead and live loads are considered to be non-reversible by nature. Hence, dead loads can belong only to loading unit 1, whereas live loads can belong only to loading units 1, 3 and 4.

3.3.1 Loading Combinations for Girders

The design quantities of a slab consist of the maximum positive and negative end and midspan moments of the four girders bordering the slab. Therefore, each girder has six design quantities consisting of the maximum positive and negative moments at ends A and B and at midspan.

For the midspan moments, the maximum moment for each loading condition is directly superimposed. It is recognized that since the location of the maximum moment is not the same for different loading conditions, the values thus obtaine are somewhat conservative.

The design quantities of the girders are obtained by finding the maximum values among the combined stress resultants due to the specified loading combinations. The first step, therefore, is to determine the combined stress resultants due to each loading combination. These consist of the maximum positive and negative moments at ends A and B of the girder, M_A^+, M_A^-, M_B^+, M_B^-, and the maximum positive and negative moments at midspan, M_C^+, M_C^-. The combined stress resultants are evaluated by accumulating the stress resultants due to the loading or loadings in each loading unit belonging to the loading combination, in accordance with the rules given in Table 9.

For a particular loading condition, the stress resultants of a girder consist of the end moments M_A and M_B, and the midspan moment M_C. When the maximum stress resultants due to live loads are evaluated, however, the equivalent of a loading combination of all the additional loading conditions has already been performed, as described in Art. 3.2.8.5. Hence, there is a value for every summing location provided for accumulating the combined stress resultants. This difference requires some additional processing, not described herein, when maximum live load effects are included in loading units 3 or 4.

3.3.2 Loading Combinations for Space Frame Columns

For the design of a column, it is desired to evaluate the critical combination of axial force and moments. Towards this end, five sets of the quantities P, M_x and M_y are accumulated for each end of the column. If only non-reversible loadings, and 'OR' combinations of reversible and non-reversible loadings occur in a loading combination (loading units 1, 3, 4 and 5), the five sets of quantities correspond to the following criteria:

1. Maximum axial force, P, on the column.
2. Maximum positive bending moment about the X axis, M_x.
3. Maximum negative M_x.
4. Maximum positive M_y.
5. Maximum negative M_y.

The stress resultants corresponding to each loading condition consist of P, M_x and M_y for each end of the column. The action taken to accumulate the above five sets of quantities can be illustrated by considering a non-reversible loading for which P, M_x and M_y are all positive. In this case P, M_x and M_y are added into set 1 because P is positive, into set 2 because M_x is positive, and into set 4 because M_y is positive.

The actions taken when loading units 1, 3, 4 or 5 occur in a loading combination are described in Table 10 with reference to the accumulation of P, M_x and M_y associated with the maximum axial force and the maximum positive value of M_x. The other sets of quantities are accumulated similarly.

When dead loads occur in a loading combination, the stress resultants P, M_x and M_y due to dead loads are added into all five sets of quantities.

When the maximum live load effect is considered, values for P, M_x and M_y corresponding to all five sets of quantities are accumulated beforehand by the equivalent of a loading combination of all the additional loading conditions. For loading unit 1, these values are added directly into the five sets. For loading units 3 and 4, which involve 'OR' combinations with a non-reversible and a reversible loading, respectively, the appropriate comparisons are made to obtain the maximum possible contributions to the five sets of quantities.

When a reversible loading (loading unit 2) occurs in a loading combination, the actions taken to accumulate the five sets of quantities are described in Table 11 for the first two sets. The other sets are accumulated similarly. For the set associated with maximum P, the loading is considered to act in the direction that produces a positive axial force, and the corresponding values of P, M_x and M_y are added into the set for each end of the column. For the remaining four sets of quantities, associated with the maximum moments, the direction of the loading for each set is considered to be the direction which produces the maximum increase in the absolute sum of the moments M_x and M_y. Thus, when reversible loadings are specified, the design quantities accumulated do not correspond exactly to the general criteria given above, but they do represent the critical combinations of axial forces and moments. Because of the criterion used, it is preferable for the reversible loadings in a loading combination to be processed after the non-reversible loadings.

3.3.3 Loading Combinations for Plane Frame Columns

The combination procedure is exactly the same as for space frame columns. However, the number of sets accumulated is different, as described below for longitudinal and lateral frames.

a) Longitudinal Frames. Three sets of the quantities P and M_y are accumulated for each end of a column. These correspond to the following criteria:

1. Maximum axial load, P.
2. Maximum positive M_y.
3. Maximum negative M_y.

b) Lateral Frames. Three sets of quantities P and M_x are accumulated for each end, corresponding to:

1. Maximum axial load, P.
2. Maximum positive M_x.
3. Maximum negative M_x.

3.4 Design Quantities

The maximum combined stress resultants obtained by considering all the loading combinations are the design quantities. As the combined stress resultants due to each specified loading combination are evaluated, they are compared with the current maxima, and the latter are updated before proceeding to the next loading combination. In this manner, when all the loading combinations have been considered, the required design quantities are contained in the current maxima. The design quantities for slabs and columns obtained via space or plane frame analyses are described below.

3.4.1 Slab Design Quantities and Design Moments

The maximum combined stress resultants of the four girders bordering a slab constitute the slab design quantities.

When a space frame analysis is performed, the maximum combined stress resultants of all the girders are obtained together, and are directly copied as the slab design quantities. On the other hand, a plane frame analysis yields only the maximum combined stress resultants of some of the girders, which are copied into the appropriate slab design quantities. The design quantities for all the slabs are therefore not obtained until all the plane frames have been analyzed.

The design quantities for a slab consist of twelve positive moments and twelve negative moments at the locations indicated in Fig. 2.7(a). Before these moments can be used to evaluate the steel required for the slabs, they are converted into column and middle strip moments in accordance with Table 2103(c) of the ACI Code. This results in positive and negative column and middle strip moments at the eighteen locations shown in Fig. 2.7(b).

In accordance with Section 2101(d) of the ACI Code, a slab is considered to consist of strips in each direction as follows: a middle strip one-half panel in width, symmetrical about the panel center line and a column strip consisting of the two adjacent quarter-panels, one on each side of the column center line.

The contribution of the design quantities to the column and middle strip moments is determined by dividing the design quantities between the two strips according to the percentages given in Table 2103(c) of the ACI Code. In the case of interior girders, the column and middle strip moments have to be divided between the two adjacent slabs in proportion to their widths perpendicular to the girder. The column and middle strip moments are further divided by the corresponding strip widths to yield the design moments per foot width of the slab.

3.4.2 Column Design Quantities

A space frame analysis yields five sets of the quantities P, M_x and M_y for each end of a column, as described in Art. 3.3.2. These quantities are the maximum quantities obtained considering all the loading combinations. However, for the purpose of design we do not need the quantities corresponding to both the maximum positive and maximum negative moments about the X and Y axes. Consequently, the five sets of quantities are reduced to three sets of design quantities for each end of a column, corresponding to the criteria:

1. Maximum axial force.
2. Maximum moment about the x axis, M_x.
3. Maximum moment about the y axis, M_y.

Set 2 is obtained by comparing the absolute values of the maximum positive M_x and the maximum negative M_x obtained as a result of all the loading combinations, and choosing the larger. Set 3 is obtained similarly.

A plane frame analysis yields three sets of quantities for each end of a column as described in Art. 3.3.3. When the maximum positive and negative moments are compared, and the set of quantities corresponding to the larger value chosen, two sets of quantities remain for each end of the column. For longitudinal frames these correspond to:

1a. Maximum axial force.

2a. Maximum moment about the Y axis, M_y.

For lateral frames the two sets of quantities correspond to:

1b. Maximum axial force.

2b. Maximum moment about the X axis, M_x.

The design quantities of a column are derived by combining the above four sets of quantities from the analyses of the longitudinal and lateral frames that the column belongs to. Two sets of design quantities are established for each end of a column. The first set, associated with the maximum P, is obtained by taking the larger of the axial forces, given by 1a and 1b, and the corresponding moments M_x and M_y. The second set is obtained by taking the maximum values of M_y and M_x from the longitudinal and lateral frame analyses and the larger axial force associated with the two moments. Therefore, the two sets of design quantities are as follows:

1. Maximum axial force, P, and M_x and M_y associated with the maximum axial force from the lateral and longitudinal analyses.

2. Maximum moment M_x, maximum moment M_y, and the larger corresponding P.

In the case of a space frame analysis the column design quantities are copied directly into the array provided for them. In the case of a plane frame analysis the array of design quantities is filled gradually as each plane frame is analyzed.

3.5 Checking of Members

The procedures employed to check the design of slabs and columns by the working stress and ultimate strength design methods are described below. These procedures are used both for checking a structure and as part of the design process.

3.5.1 Checking of Slabs

The checking procedures for singly and doubly reinforced sections are presented below.

3.5.1.1 Checking of Singly Reinforced Sections

a) Working Stress Method. Figure 3.12 shows a singly reinforced section. The effective depth of the section, d, is given by:

$$d = d_t - d_c - \frac{3}{8} \tag{3.35}$$

where d_t is the total depth of the slab and d_c is the minimum clear cover for the steel. The input data includes the values of the minimum top and bottom clear covers, and the appropriate value is used depending upon the direction of the applied moment. The additional $\frac{3}{8}$ inch represents an approximation for half the diameter of the main reinforcing bars.

Consider a one foot wide strip of the slab. If A_s is the area of tensile steel provided per foot of width, the depth of the neutral axis is located by:

$$k = \sqrt{2pn+(pn)^2} - pn \tag{3.36}$$

where $p = A_s/12d$, and n is the modular ratio E_s/E_c. The lever arm is given by:

$$jd = d\left(1 - \frac{k}{3}\right) \tag{3.37}$$

and the tensile steel stress, f_s, by

$$f_s = \frac{M}{A_s jd} \tag{3.38}$$

where M is the applied moment acting at the section. Finally, the extreme fiber concrete stress, f_c, is obtained from:

$$f_c = \frac{f_s}{n}\left(\frac{kd}{d-kd}\right) \tag{3.39}$$

b) Ultimate Strength Method. The section is checked in accordance with the provisions of Section 1601 of the ACI Code.

The effective depth of the section is determined from Eq. (3.35), and the reinforcement ratio, p_b, which produces balanced conditions at ultimate strength is evaluated from:

$$p_b = \frac{0.85k_1 f'_c}{f_y} \frac{87{,}000}{87{,}000+f_y} \tag{3.40}$$

where f'_c is the compressive strength of the concrete, f_y is the yield strength of the reinforcement, and k_1 is specified in Section 1503(g) of the ACI Code as:

$$k_1 = 0.85-0.05(f'_c-4000) \leq 0.85 \tag{3.41}$$

Considering a one foot wide strip of the slab, the reinforcement ratio p is equal to:

$$p = \frac{A_s}{12d} \tag{3.42}$$

A check is made to ascertain that p does not exceed $0.75p_b$, as required by the code. If $p \leq 0.75p_b$, the ultimate resisting moment of the section is evaluated from

$$M_r = \Phi[A_s f_y\left(d - \frac{a}{2}\right)] \tag{3.43}$$

in which the coefficient Φ is equal to 0.90, and a is given by:

$$a = \frac{A_s f_y}{0.85f'_c b} = \frac{A_s f_y}{10.2f'_c} \tag{3.44}$$

3.5.1.2 Checking of Doubly Reinforced Sections

a) Working Stress Method. The semielastic method(4) is used to check the section. The positive and negative applied moments are considered in turn, the positive moment, M_p, being considered first.

Referring to Fig. 3.13, the effective depth d and the top cover d' of the section are equal to

$$d = d_t - d_{bc} - \frac{3}{8} \qquad (3.45)$$

$$d' = d_{tc} + \frac{3}{8} \qquad (3.46)$$

in which d_{tc} and d_{bc} represent the top and bottom clear covers for the steel.

Consider a one foot wide strip of the slab, and let A_s and A_s' represent the tensile and compressive steel areas provided per foot of width. Assuming that the effective modular ratio for the compression reinforcement is 2n and that the compressive steel stress, f_s', is less than the allowable stress, f_{sa}, the neutral axis is located by:

$$k = \sqrt{2(2n-1)p'\frac{d'}{d} + 2pn + n^2(2p'+p - \frac{p'}{n})^2} - n(2p'+p - \frac{p'}{n}) \qquad (3.47)$$

where $p = A_s/12d$ and $p' = A_s'/12d$. The lever arm is given by:

$$jd = d - \frac{(d/6)k^2 + p'(2n-1)(1-d'/kd)d'}{(k/2)+p'(2n-1)(1-d'/kd)} \qquad (3.48)$$

The tensile and compressive steel stresses are given by:

$$f_s = \frac{M_p}{A_s jd} \qquad (3.49)$$

$$f_s' = 2f_s \frac{kd-d'}{d-kd} \qquad (3.50)$$

and the extreme fiber concrete stress is given by Eq. (3.39).

If f_s' is greater than f_{sa}, Eqs. (3.47) to (3.50) are invalid, because they are based upon the assumption $f_s' < f_{sa}$. The following procedure is then used to determine the steel and concrete stresses. The compressive steel stress f_s' is set equal to f_{sa}, and f_c is determined by taking moments about the tensile steel as:

$$f_c = \frac{M_p - f_{sa}(d-d')}{6kd(d - \frac{kd}{3}) - (\frac{kd-d'}{kd})(d-d')} \tag{3.51}$$

In Eq. (3.51) kd is unknown, but, as a first approximation, the value of kd given by Eq. (3.47) is used. An improved value for the depth of the neutral axis is now obtained, by equating the total compressive and tensile forces, as the root of the quadratic equation:

$$kd = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \tag{3.52}$$

in which $a = 6f_c$

$b = A_s'(f_{sa} - f_c) + A_s nf_c$

$c = f_c(A_s'd' - A_s nd)$

After evaluating kd using Eq. (3.52), Eqs. (3.51) and (3.52) are repeated once to improve upon the approximation for kd, and then f_s is determined from

$$f_s = nf_c \frac{d-kd}{kd} \tag{3.53}$$

When the steel and concrete stresses due to the positive moment have been computed, the negative moment acting at the section, M_n, is considered. For a negative moment, the values of d and d' are

$$d = d_t - d_{tc} - \frac{3}{8} \tag{3.54}$$

$$d' = d_{bc} + \frac{3}{8} \tag{3.55}$$

and, since the direction of the moment is reversed, the previous tensile steel, A_s, acts as the compressive steel, A_s', and vice versa. Using the new values for d, d', A_s and A_s', and M_n in place of M_p, the procedure given above is repeated to yield the steel and concrete stresses due to the negative moment.

b) Ultimate Strength Method. The section is checked in accordance with the provisions of Section 1602 of the ACI Code.

The positive and negative moments acting at the section are considered in turn. For the positive moment, M_p, the effective depth d and the top cover d' of the section are computed from Eqs. (3.45) and (3.46). Before the resisting moment of the section can be evaluated, it is necessary to check whether the compression steel yields at ultimate strength, i.e., whether

$$p-p' \geq 0.85k_1 \frac{f'_c d'}{f_y d} \frac{87,000}{87,000-f_y} \tag{3.56}$$

When the inequality (3.56) is not satisfied, the ultimate resiting moment of the section is computed as for a singly reinforced section, and is given by Eq. (3.43).

When (p-p') satisfies the inequality (3.56), the code stipulates that (p-p') must not exceed 0.75 times the value of p_b given by Eq. (3.40). If this condition is satisfied, the ultimate resisting moment of the section is given by

$$M_r = \Phi[(A_s-A'_s)f_y(d - \frac{a}{2})+A'_s f_y(d-d')] \tag{3.57}$$

where Φ is equal to 0.90, and a is given by

$$a = (A_s-A'_s) \frac{f_y}{0.85f'_c b} = (A_s-A'_s) \frac{f_y}{10.2f'_c} \tag{3.58}$$

For the negative moment acting at the section, M_n, the effective depth and top cover of the section are given by Eqs. (3.54) and (3.55). Using these values of d and d', and reversing the roles of A_s and A_s', the negative ultimate resisting moment is evaluated in the same manner as the positive moment.

3.5.2 Checking of Columns

The first step in the column checking procedure is the determination of the effective column length and the corresponding strength reduction factor. Next, the column design quantities are increased to account for the strength reduction, and the column is then checked using the appropriate formulas for short columns given in Chapters 14 and 19 of the ACI Code for the design of columns by the working stress and ultimate strength design methods, respectively.

3.5.2.1 Determination of Effective Length

The effective length of a column is determined as per Section 915 of the ACI Code. In the case of analysis alternatives in which sidesway is not considered, the effective length, h', is equal to the actual length h of the column. When sidesway is permitted, the end conditions of the column are determined by evaluating r', which is defined as the ratio of $\sum \frac{EI}{L}$ of the columns to $\sum \frac{EI}{L}$ of the floor members in a plane at one end of the column. The value of r' is evaluated for both planes and then averaged at each end. If the average value of r' is greater than 25 at either end, the end is considered to be hinged, otherwise it is regarded as being restrained against rotation. For columns restrained against rotation at one end and hinged at the other end, the effective length is given by

$$h' = 2h(0.78+0.22r') \geq 2h \tag{3.59}$$

where r' is the value at the restrained end. For columns restrained against rotation at both ends, the effective length is given by

$$h' = h(0.78+0.22r') \geq h \tag{3.60}$$

where r' is the average of the values at the two ends of the column.

3.5.2.2 Strength Reduction Due to Length

In order to compensate for the strength reduction of a column due to its length, the column design quantities are increased as recommended in Section 916 of the ACI Code. It is assumed, however, that the design of the columns is governed by compression. If, in fact, a section is governed by tension, the increased design quantities used will be somewhat conservative, which is considered acceptable.

For the analysis alternatives in which sidesway is neglected, the reduction factor by which the column design quantities are divided is given by

$$R = 1.07-0.008\,\frac{h}{r} \leq 1.0 \tag{3.61}$$

in which h is the length of the column and r is the radius of gyration of the gross concrete section. For the square columns considered, r is equal to $0.3t$, where t is the side dimension.

When sidesway is not prevented, the reduction factor is obtained by using the effective length of the column, h', in place of h in Eq. (3.61). Furthermore, the ACI Code specifies that when the design is governed by lateral loads of short duration such as wind or earthquake loads, the factor R may be increased by 10 percent. To take this provision into account, it is assumed in the procedure that the maximum axial load on the columns is caused by the vertical loading conditions, whereas the maximum moments are caused by the lateral loads. Hence, the increased reduction factor is used only for the design quantities associated with the maximum moments M_x and M_y.

3.5.2.3 Checking of Column Section

The column section is checked separately for each of the six sets of design quantities, appropriately increased to account for the strength reduction due to column length. Considering a single set of design quantities P, M_x and M_y, the equivalent eccentricities of the axial load $e_x = M_x/P$ and $e_y = M_y/P$ are evaluated. In accordance with Section 1901(a) of the ACI Code the minimum eccentricity for tied columns about either principal axis should be 0.1t. Therefore, the calculated eccentricities are compared against 0.1t and increased if necessary. The eccentricities e_x and e_y are next compared with the balanced eccentricity for the section, e_b. If both the eccentricities e_x and e_y are less than or equal to the balanced eccentricity, the column is considered to be controlled by compression, otherwise it is regarded as being tension controlled.

The formula for the balanced eccentricity, and the relations that must be satisfied for compression controlled and tension controlled sections in the working stress and ultimate strength methods are described below.

a) Working Stress Method. In accordance with Section 1407 of the ACI Code, the balanced eccentricity is given by

$$e_b = (0.67p_g m+0.17)d \tag{3.62}$$

where p_g is the ratio of the longitudinal reinforcement to the gross area of the section A_g, m is equal to $f_y/0.85f'_c$ and d is the effective depth of the section given by

$$d = t - d' \tag{3.63}$$

where d' is the distance from the face of the column to the center of the vertical reinforcement, as shown in Fig. 3.14. The value of d' is approximated by

$$d' = d_{cc} + 1.0 \tag{3.64}$$

where d_{cc} is the clear cover for the steel specified in the input, and the additional 1.0 inch represents an approximation for the thickness of the lateral ties plus half the diameter of the vertical reinforcement.

For columns controlled by compression, Section 1407 of the ACI Code specifies that the following inequality must hold:

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b} \leq 1.0 \tag{3.65}$$

where f_a represents the axial force divided by the gross area of the section, f_{bx} and f_{by} are the bending moment components about the X and Y principal axes divided by the section modulus of the respective transformed uncracked section (2n being assumed as the modular ratio for all vertical reinforcement), F_b is the allowable bending stress f_{ca}, and

$$F_a = 0.34(1+p_g m)f'_c \tag{3.66}$$

In order to evaluate the transformed moment of inertia and, hence, the required section modulus, it is assumed that the vertical reinforcement is evenly spaced around the periphery of the column. When the vertical reinforcement consists of four bars at the corners, the transformed moment of inertia of the steel about either principal axis is given by

$$I_s = (2n-1)A_{st}d_h^2 \tag{3.67}$$

in which 2n is the modular ratio, A_{st} is the total area of longitudinal steel and

$$d_h = \frac{t}{2} - d' \tag{3.68}$$

When the vertical reinforcement consists of more than four bars the transformed moment of inertia is given by

$$I_s = k_r(2n-1)A_{st}d_h^2 \tag{3.69}$$

in which k_r is equal to 0.75 for 8 bars, 0.70 for 12 bars, 0.69 for 16 bars, and 0.68 for 20 or 24 bars.

When the column is tension controlled, the inequality that must be satisfied is specified by Section 1407 of the ACI Code as:

$$\frac{M_x}{M_{ox}} + \frac{M_y}{M_{oy}} \leq 1.0 \tag{3.70}$$

in which M_{ox} and M_{oy} are the values of M_o for bending about the X and Y principal axes, where M_o is the allowable bending moment when the section is in pure flexure. For symmetrical tied columns M_o is given by

$$M_o = 0.40A_sf_y(d-d') \tag{3.71}$$

in which A_s is the area of the tension reinforcement.

For each set of design quantities, the inequality (3.65) is checked if the column is compression controlled, and (3.70) is checked if the column is tension controlled. If either inequality is not satisfied, the column section is considered to be unsafe.

b) <u>Ultimate Strength Method</u>. The balanced load for the column section is given by Eq. (19-1), Section 1903, of the ACI Code as

$$P_b = \Phi[0.85f_c'ta_b+(A_s'-A_s)f_y] \tag{3.72}$$

in which Φ is recommended by Section 1504 of the Code to be equal to 0.70 for tied compression members, t is the width of the column, A_s and A_s' are the areas of tension and compression reinforcement, and a_b is the depth of the equivalent rectangular stress block for balanced conditions, given by:

$$a_b = k_1 c_b = k_1 d \frac{87{,}000}{87{,}000+f_y} \tag{3.73}$$

where c_b is the distance from the extreme compression fiber to the neutral axis for balanced conditions.

The balanced eccentricity for the column section is obtained from Eq. (19-3) of the ACI Code as:

$$e_b = \frac{\Phi[0.85f_c' t a_b (d-d''-\frac{a_b}{2})+A_s' f_y (d-d'-d'')+A_s f_y d'']}{P_b} \tag{3.74}$$

where d is the distance from the extreme compression fiber to the centroid of the tension reinforcement, d' is the distance from the extreme compression fiber to the centroid of the compression reinforcement, and d'' is the distance from the plastic centroid to the centroid of the tension reinforcement.

The ACI Code does not make any specific recommendation for the treatment of columns subject to biaxial loads. In this study, the ultimate strength of the column, P_{ult}, is calculated from the empirical formula given by Bresler[2] as:

$$\frac{1}{P_{ult}} = \frac{1}{P_x} + \frac{1}{P_y} - \frac{1}{P_o} \tag{3.75}$$

where P_x and P_y represent the uniaxial load capacities under eccentric loads along the Y and X axes, respectively, and P_o represents the load capacity of the column in pure axial compression, given by Eq. (19-7) of the ACI Code as:

$$P_o = \Phi[0.85f_c'(A_g-A_{st})+A_{st}f_y] \tag{3.76}$$

The values of P_x and P_y depend upon whether the section is controlled by tension or compression. When the section is tension controlled, i.e., the equivalent eccentricity of the axial load is greater than the balanced eccentricity, the uniaxial load capacity of symmetrically reinforced columns is given by Eq. (19-5) of the ACI Code as:

$$P_u = \Phi[0.85f_c'td\{-p+1-e'/d+\sqrt{(1-e'/d)^2+2p[m'(1-d'/d)+e'/d]}\}] \tag{3.77}$$

where $p = A_s/td$, e' is the eccentricity of the axial load from the centroid of the tension reinforcement, and m' is given by:

$$m' = \frac{f_y}{0.85f_c'} - 1 \tag{3.78}$$

When the section is controlled by compression, the uniaxial load capacity of the column is evaluated from Eq. (19-8) of the ACI Code as:

$$P_u = \frac{P_o}{1+[(P_o/P_b)-1]e/e_b} \tag{3.79}$$

where e is the eccentricity of the axial load from the plastic centroid of the section.

For each set of design quantities, P_{ult} is evaluated using Eq. (3.75) and compared with the axial load P acting on the column. The column section is considered to be safe is

$$\frac{P}{P_{ult}} \leq 1.0 \tag{3.80}$$

3.6 Design

Ideally, the design process should consist of an iterative loop around the member checking procedures, irrespective of the design method used. However, in the existing working stress and ultimate strength design procedures, the parameters that can be varied to achieve a satisfactory design and the tests required are different. Therefore, even though the two design methods are very similar, it was found more convenient to develop separate design algorithms for the two methods, as described below.

3.6.1 Design of Slabs

The design of a slab consists of determining the depth of the slab and the amount and location of steel reinforcement required. A description of the determination of slab depths is followed by a description of the design of singly and doubly reinforced sections by the working stress and ultimate strength methods of design.

3.6.1.1 Determination of Slab Depths

Unless a slab has been specified in the input to be of a fixed depth, the thickness of the slab is determined empirically. In accordance with Section 2104(d) of the ACI Code, the slab thickness should be the minimum of $L'/36$, 5 inches, or d_t given in inches by:

$$d_t = 0.028L'\left(1 - \frac{2c}{3L'}\right)\sqrt{\frac{w'}{f'_c/2000}} + 1\frac{1}{2} \qquad (3.81)$$

where L' is the longer side of the panel in inches, c is the width in inches of the supporting columns in the direction considered, w' is the uniformly distributed unit dead plus live load in psf, and f'_c is the compressive strength of concrete in psi. Since the slab dead load is not known initially, it is assumed to be 60 psf plus the superimposed dead load. Using this as an initial value, Eq. (3.81) is iterated upon twice using the improved value of d_t to determine w' each time. The width of the supporting columns, c, is set equal to the conservative value of 12 inches.

3.6.1.2 Design of Singly Reinforced Sections

A section is designed as a singly reinforced section, as shown in Fig. 3.12, if both the design moments at the section are of the same sign. The section is designed for the numerically larger moment, M.

a) Working Stress Method. A trial value for the area of tensile steel is evaluated from

$$A_s = \frac{M}{f_{sa} jd} \tag{3.82}$$

where jd is approximated by $\frac{7}{8}$d. The steel stress, f_s, is evaluated using Eqs. (3.36) to (3.38), and compared with the allowable stress. If $f_s > f_{sa}$ a new trial value for the area of steel, A_{sn}, is obtained from

$$A_{sn} = A_s \frac{f_s}{f_{sa}} \tag{3.83}$$

and, using A_{sn} in place of A_s, Eqs. (3.36) to (3.38) are repeated until $f_s < f_{sa}$. When $f_s < f_{sa}$, f_c is evaluated using Eq. (3.39) and compared with the allowable stress, f_{ca}. If $f_c > f_{ca}$, it is recognized that a singly reinforced section is inadequate for the applied moment and the section is designed as a doubly reinforced section, using A_s for the trial value of the tensile steel, and for the compressive steel:

$$A_s' = 11kd\,\frac{f_c - f_{ca}}{f_{sa}} \tag{3.84}$$

The value of A_s' in Eq. (3.84) is an empirical relation derived from an estimate of the compressive force that should be taken by the compressive steel in order to reduce the concrete stress to an acceptable value.

If, however, it is found that $f_c < f_{ca}$, the values of f_c and f_s are compared against 0.95 f_{ca} and 0.95 f_{sa} respectively. If both $f_c < 0.95\ f_{ca}$ and $f_s < 0.95\ f_{sa}$, the steel area is reduced in accordance with Eq. (3.83) and the checking process is repeated. When either or both $f_c > 0.95\ f_{ca}$ and $f_s > 0.95\ f_{sa}$, the design is considered satisfactory.

b) Ultimate Strength Method. The procedure is essentially the same as for the working stress method, except that the ultimate resisting moment of the section is compared with the applied moment.

A trial value for the area of tensile steel is evaluated from

$$A_s = \frac{M}{f_y jd} \tag{3.85}$$

where jd is approximated by $\frac{7}{8}$d. Using the trial value for A_s, the section is checked as described in Art. 3.5.1.1(b), and the ultimate resisting moment M_r is evaluated using Eq. (3.43). If M_r is found to be less than the applied moment M, a new trial value for the area of steel, A_{sn}, is evaluated from

$$A_{sn} = A_s \cdot \frac{M}{M_r} \tag{3.86}$$

and, using A_{sn} in place of A_s, the checking procedure is repeated until $M_r \geq M$. If at any stage of iteration, the reinforcement ratio p exceeds

0.75 p_b, where p_b is given by Eq. (3.40), it is recognized that a singly reinforced section is inadequate and the section is designed as a doubly reinforced section with 0.75 p_b as the trial value for A_s, and the conservative value of 0.05 sq. in. for the compression reinforcement.

When $M_r > M$, the design moment is compared against 0.95 M_r to check the efficiency of the design. If $M < 0.95M_r$, the steel area is reduced in accordance with Eq. (3.86), and the design process is repeated.

3.6.1.3 <u>Design of Doubly Reinforced Sections</u>

A section is designed as a doubly reinforced section, as shown in Fig. 3.13, when the two design moments at the section are of opposite signs or when a singly reinforced section is found to be inadequate.

a) <u>Working Stress Method</u>. The two applied moments are considered separately, the positive moment M_p being considered first. If no trial values are available from an attempted singly reinforced design, a trial value for the tensile steel A_s is obtained by using M_p in place of M in Eq. (3.82), and a trial value for the compressive steel is obtained from

$$A_s' = \frac{M_n}{f_{sa} jd} \tag{3.87}$$

where M_n is the negative moment acting at the section, and jd is approximated by $\frac{7}{8}$ $(d_t - d')$.

Assuming that $f_s' < f_{sa}$, the actual lever arm is now determined using Eqs. (3.47) and (3.48), and f_s is computed from Eq. (3.49) and compared with the allowable stress f_{sa}. If $f_s > f_{sa}$, a new trial value for the area of steel, A_{sn}, is computed from Eq. (3.83) and, using A_{sn} in place of A_s, Eqs. (3.47) to (3.50) are repeated until $f_s < f_{sa}$.

When $f_s < f_{sa}$, the extreme fiber concrete stress is evaluated using Eq. (3.39) and is compared with the allowable stress f_{ca}. If $f_c > f_{ca}$, the area of compression reinforcement is increased to a new trial value A'_{sn} using:

$$A'_{sn} = mA'_s \frac{f_c}{f_{ca}} \tag{3.88}$$

where m is an empirical factor which is varied for rapid convergence as follows:

For $A'_s \leq 0.5$ sq. ins., $m = 2.0$

For $A'_s \leq 1.0$ sq. ins., $m = 1.5$

For $A'_s > 1.0$ sq. ins., $m = 1.25$

The new value for the compression reinforcement, A'_{sn}, is used in place of A'_s and Eqs. (3.47) to (3.50) are repeated until $f_c < f_{ca}$. When $f_c < f_{ca}$, the compressive steel stress is compared with the allowable stress. If $f'_s > f_{sa}$, the calculated stresses f_c and f_s are invalid, because they are based upon the assumption $f'_s < f_{sa}$. The compressive steel stress is then set equal to f_{sa}, and f_c is computed from Eq. (3.51). If f_c is found to be greater than f_{ca}, the area of compression reinforcement is increased in accordance with Eq. (3.88), and the whole design process is repeated. When $f_c < f_{ca}$, the tensile steel stress is determined using Eqs. (3.51) to (3.53), as described in Art. 3.5.1.2(a). If the evaluated stress f_s is greater than the allowable stress, the area of tensile steel is increased in accordance with Eq. (3.83) and the design process is repeated.

When all the stresses due to the positive moment are equal to or less than the allowable values, the design procedure is repeated for the negative moment to yield updated values for A_s and A'_s.

The acceptability of the design is checked as follows. Denoting by f_{c1} and f_{s1} the concrete and tensile steel stresses due to M_p, and by f_{c2} and f_{s2} the same quantities due to M_n, f_{c1} and f_{c2} are compared against $0.95f_{ca}$, while f_{s1} and f_{s2} are compared against $0.95f_{sa}$. If at least one of the stresses is greater than 0.95 times the allowable stress, the design is accepted. Otherwise f_{s1} is compared with f_{s2} and the following actions taken:

$$\text{If } f_{s1} \leq f_{s2} \qquad A_{sn} = A_s \frac{f_{s1}}{f_{sa}}$$

$$\text{If } f_{s1} > f_{s2} \qquad A'_{sn} = A'_s \frac{f_{s2}}{f_{sa}}$$

Using the new trial values for the reinforcement, the design procedure is repeated for both the positive and negative moments until an acceptable design is attained.

(b) Ultimate Strength Method. The positive moment, M_p, acting at the section is considered first. In the absence of previous trial values, a trial value for A_s is obtained by using M_p in place of M in Eq. (3.85), and a trial value for A'_s is obtained from

$$A'_s = \frac{M_n}{f_y jd} \tag{3.89}$$

where jd is taken as $\frac{7}{8}(d_t - d')$.

The trial section is checked as described in Art. 3.5.1.2(b). If (p-p') is found to be greater than $0.75p_b$, A'_s is reduced such that

$$p - p' = 0.75p_b. \tag{3.90}$$

A check is now made to determine whether the inequality (3.56) is satisfied. If it is, the ultimate resisting moment of the section M_r is evaluated from Eq. (3.57).

If Eq. (3.56) is not satisfied, the value of p is checked against $0.75p_b$ and the following actions taken. If $p \leq 0.75p_b$, M_r is computed as a singly reinforced section from Eq. (3.43). If $p > 0.75p_b$, p is set equal to $0.75p_b$ and M_r is then computed from Eq. (3.43). If, however, the value of M_r given by Eq. (3.43) is inadequate, A_s is adjusted such that the inequality (3.56) is satisfied and M_r is evaluated from Eq. (3.57).

After M_r has been evaluated, it is compared with M_p. If $M_r < M_p$, the area of tensile steel is increased in accordance with Eq. (3.86), and the checking procedure is repeated until $M_r \geq M_p$.

When $M_r \geq M_p$, the entire design procedure is repeated for the negative moment, M_n.

In order to check the efficiency of the design, letting M_{rp} and M_{rn} represent the positive and negative ultimate resisting moments, respectively, M_p is compared with 0.95 M_{rp} and M_n with 0.95 M_{rn}. If at least one of the applied moments M_p or M_n is greater than 0.95 times the resisting moment, the design is accepted. Otherwise, M_p/M_{rp} is compared with M_n/M_{rn} and the following actions taken:

$$\text{If } \frac{M_p}{M_{rp}} \leq \frac{M_n}{M_{rn}} \qquad A_{sn} = A_s \frac{M_p}{M_{rp}}$$

$$\text{If } \frac{M_p}{M_{rp}} > \frac{M_n}{M_{rn}} \qquad A'_{sn} = A'_s \frac{M_n}{M_{rn}}$$

Using the new trial values for the reinforcement, the design procedure is repeated until an acceptable design is obtained.

3.6.2 Design of Columns

The design of a column consists of checking an assumed section and modifying it, if necessary, until the desired efficiency is attained.

3.6.2.1 Checking and Modification of Column Sections

The trial concrete area of a column is obtained from the input trial side dimension. The trial steel area is calculated from the input trial steel percentage times the concrete area, and then rounded up to the nearest value found in a built-in table of steel areas. This table of areas corresponds to 4, 8, 12, 16, 20 or 24 bars ranging from #5 to #11 bars. The bars are assumed to be placed symmetrically around the four sides of the column.

The trial section is checked for each of the six sets of design quantities, appropriately increased for the strength reduction due to column length. The checking procedure is described in Art. 3.5.2.3. In the working stress method, the inequality (3.65) is checked if the column is compression controlled, and (3.70) is checked if the column is tension controlled. In the ultimate strength method, the inequality (3.80) is checked.

In either method, if the applicable inequality is not satisfied the trial section is revised as follows. First, an attempt is made to increase the steel area until the appropriate inequality is satisfied. If the steel area cannot be increased without exceeding the maximum allowable steel percentage, the side dimension of the column, t, is increased by one inch and the area of steel made as close as possible to the maximum allowable steel percentage.

The checking procedure is repeated and the column section gradually revised until all six sets of design quantities satisfy inequality (3.65), (3.70) or (3.80). A measure of the efficiency of the design is retained by storing the value of the left hand sides of inequalities (3.65), (3.70) or (3.80) for each of the six sets of design quantities.

3.6.2.2 Reducing of Overdesigned Sections

The efficiency of the designed section is assessed by comparing the stored "efficiency factors" against 0.95. If any one of the six factors is greater than 0.95, no attempt is made to reduce the section. If, on the other hand, all six factors are less than 0.95 an effort is made to reduce the section. In order to avoid getting caught in a loop, however, if the steel or concrete area of a section has been increased during the checking procedure, suitable flags are set so that no attempt is made to reduce the corresponding quantities even when the efficiency is below 95 percent.

The procedure employed to reduce the section is as follows. If the concrete area of the column has not been increased during the checking phase, the column side dimensions are reduced by one inch and the new trial value for the area of steel is set as close as possible to the maximum allowable steel percentage. The checking procedure is then repeated for the new trial section. If, on the other hand, the concrete area has been increased but the steel area has not, an attempt is made to reduce the steel area in proportion to the largest efficiency factor. The reduced value is then rounded up to the nearest value in the built-in table of steel areas. If it is found that the area of steel remains unchanged, it is assumed that the section cannot be reduced any further. If the area of steel is in fact reduced, the checking procedure is repeated for the new trial section.

The above process is continued until the section has been refined to such an extent that neither the concrete nor the steel area can be further reduced.

3.7 Design of Member Groups

The data pertaining to member groups are read in as part of the input data and stored for later use during the design phase. The description of the storage is followed by a description of the design of member groups.

3.7.1 Storage of Group Information

a) Slabs. The three types of slab groups incorporated into the model, namely, slabs with fixed depth, slabs with equal depth and identical slabs, are described in Art. 2.6.2.

Three memory locations are reserved per slab corresponding to the three types of groups. When a slab belongs to one of the slab groups specified, the group number is recorded in the appropriate location corresponding to that slab. In addition, for slabs of fixed depth, the depth as input is stored in a location corresponding to the group number. Note that it is possible for a slab to belong to a group of slabs with equal depth as well as to a group of identical slabs.

b) Columns. Two categories of column groups are included in the model, as described in Art. 2.6.2.

The data pertaining to the first category, i.e., preassigned groups, are stored in two locations per column. If a column belongs to a group of columns of equal dimensions, the group number is stored in the first location. If it belongs to a group of identical columns, the corresponding group number is stored in the second location. Along with the data pertaining to every group of columns of the first category, a trial size is also read in and stored according to the group number. This trial size is used to determine the member properties and the self-weight of the columns. For all columns that do not fall into any group, a single common size is input and used as the first trial value.

The second category consists of program selected groups of columns based upon specified constraints. The constraint adopted in the present model is the area of the columns. The information that has to be stored for each group is the type of group, i.e., a group of columns of equal dimensions (Type 1) or an identical group (Type 2), and the lower and upper limits prescribed. Three memory locations per group are used to store the pertinent data.

3.7.2 Determination of Slab Depths

Before the thickness of a slab is evaluated, it is checked to determine whether the slab belongs to any group of slabs of fixed depth. If it does, the corresponding group depth is assigned to the slab. If the slab does not belong to any group of slabs of fixed depth, the required slab thickness is evaluated as described in Art. 3.6.1.1. Next, the slab is checked to determine whether it belongs to any group of slabs with equal depth or identical slabs. If it does, the tentative control depth of the group is compared against the calculated slab thickness. If the calculated thickness of the slab is greater than the control depth, the control depth is made equal to the calculated depth. In this manner, after all slab thicknesses have been determined, the control depth for each group is equal to the required slab depth for that group. A second pass is now performed to set the depth of all slabs that belong to groups to the corresponding control depths.

3.7.3 Determination of Slab Steel

The steel required at every section is determined as described in Art. 3.6.1.2. Each of the eighteen slab design sections are considered separately, and the steel required at a particular section is determined for all the slabs in the structure before proceeding to the next section. This sequence, rather than the more obvious one of determining the steel required at all eighteen sections of a slab before proceeding to the next slab, was chosen because it requires much less storage for handling identical slab groups.

For slabs that do not belong to any group of identical slabs, the areas of steel evaluated are the final quantities. In the case of slabs that belong to groups of identical slabs, on the other hand, a procedure similar to that described for the determination of slab thicknesses is implemented. After the top and bottom steel areas required at a particular section of a slab are determined, they are compared with the control areas for the group that the slab belongs to. If the control areas are smaller, the control areas are set equal to the new values. When all the slabs have been processed, the control areas are equal to the maximum areas of steel required for each group of identical slabs. A second pass is now required to update all the slabs that belong to groups of identical slabs by making the steel areas equal to the appropriate control areas.

3.7.4 Determination of Column Sections

For each column, the trial section is first checked and modified, as described in Art. 3.6.2.1. If the column belongs to a preassigned group of columns, its concrete area is compared with the current control area for the group that it belongs to. If the concrete area of the column is greater than the current control area, or if the column does not belong to any member group, an attempt is made to reduce the section as described in Art. 3.6.2.2. After any possible reduction has been made in the column section, the control areas of the group or groups (if any) that it belongs to are updated before proceeding to the checking and modification of the next column.

When all the columns have been processed, the concrete and steel areas of all columns belonging to preassigned groups are updated as follows. The concrete areas are made equal to the corresponding control areas. For identical columns, the steel areas are also made equal to the control areas, whereas for columns of equal dimensions, an attempt is made to reduce the steel area if the concrete area has been increased during the updating pass.

After all the columns have been updated, the areas of all the columns are compared against the lower and upper limits specified for the selection of program-selected groups. If the area of a column falls between any set of lower and upper limits, the column is assigned to the corresponding group and the control areas of the group are updated, if necessary.

After the columns that fall into program-selected groups, as well as the control areas for each group, have been determined, the concrete and steel areas are updated again in a manner identical to that described above for preassigned groups.

It is conceivable that a column which falls into a program-selected group of columns with equal dimensions may have already been specified by the designer to be a member of a group of identical columns. This obvious anomaly is removed by determining the new control areas of steel for all groups of identical columns and setting the steel areas equal to the control areas.

3.8 Design on Basis of Partial Analysis

As described in Art. 2.6.3, it is possible to design a structure crudely by analyzing only a few of the plane frames instead of all the plane frames or the complete space frame. The design procedure for slabs and columns is described separately below.

3.8.1 Slabs

Since all the plane frames have not been analyzed, the design quantities are not complete. For slabs that do not belong to any member group or belong to a group of slabs of equal depth, the missing design quantities, if any, are approximated on the basis of the available quantities in the manner described below. For slabs that belong to groups of identical slabs, approximate quantities are created only for the control slabs of each group. The control slabs of a group are defined as those slabs in the group for which the largest number of design quantities is available. Finding the control slabs of a group amounts to determining the slabs with the maximum number of sides that fall into the analyzed plane frames.

The determination of the approximate design quantities is best described with reference to the slab shown in Fig. 3.15. Suppose that the design quantities corresponding to side 2 are not available. If the design quantities for side 1, the opposite side, are available, the missing quantities are approximated by those of side 1. If the design quantities for side 1 are not available, the missing quantities are approximated by the design quantities of sides 3 or 4, whichever available, modified as follows:

$$D_q^1 = D_q^3 \left(\frac{\ell_1}{\ell_2}\right)^2 \text{ or } D_q^1 = D_q^4 \left(\frac{\ell_1}{\ell_2}\right)^2 \tag{3.91}$$

in which D_q^k represents the design quantities of side k, and ℓ_1 and ℓ_2 are the lengths of the sides, as shown in Fig. 3.15. In other words, the missing design quantities of a slab are approximated by those of the opposite side without change, or by those of one of the perpendicular sides multiplied by the ratio of the side lengths squared, as indicated in Eq. (3.91).

After all the missing design quantities have been created, the design quantities are transformed into column and middle strip moments as described in Art. 3.4.1. Next, the required steel is determined, as described in Art. 3.6.1, processing only the control slabs among the slabs belonging to the groups of identical slabs, while processing all the other slabs in the structure. The control areas of steel for each group of identical slabs are determined in the first pass from the steel areas of the control slabs, and in the second pass the steel areas for all the slabs belonging to the groups of identical slabs are set equal to the control areas.

3.8.2 Columns

The procedure for handling columns on the basis of a partial analysis is very similar to that described for slabs. For all the columns that do not belong to any member group or belong to a group of columns of equal concrete dimensions, any missing design quantities are approximated as described below. For columns that belong to groups of identical columns, fictitious quantities (if required) are created only for those columns in each group for which the most design quantities are already available. These are termed control columns. All the design quantities of a column are determined if both the longitudinal and lateral plane frames it is part of have been analyzed. Hence, the control columns are determined by finding the columns that are part of the maximum number (1 or 2) of plane frames analyzed.

A column for which fictitious design quantities have to be created must belong to one analyzed frame. The missing design quantities are simply set equal to the corresponding quantities obtained from the plane frame analyzed.

After all the missing design quantities have been created, the columns are designed as described in Art. 3.6.2. Among the columns belonging to groups of identical columns, only the control columns are designed, and the control areas of concrete and steel for these groups are determined during the process. After the initial design of the columns, they are updated as described in Art. 3.7.4. During the updating process, the column and steel areas of all columns belonging to groups of identical columns are set equal to the corresponding control areas.

Program-selected groups of columns are unacceptable when a partial analysis is performed.

3.9 Checking of a Structure

When the model is used to check a previously designed structure, the actual member sizes and steel areas provided are part of the input. The input sizes are used to determine the elastic properties following which the structure is analyzed for the loadings specified, and the member design quantities are evaluated considering the specified loading combinations. Whether the structure is being designed or checked, there is absolutely no difference in the analysis and the evaluation of design quantities. In contrast, the member checking procedures given in Art. 3.5 are applied directly rather than iteratively.

a) Slabs. The column and middle strip moments are used to check the corresponding sections. If the checking is done by the working stress method, the steel and concrete stresses due to the applied moments are computed and compared with the allowable stresses. If the checking is performed by the ultimate strength method, the resisting moments are computed and compared with the applied moments. If at any section the allowable stresses are exceeded or the resisting moment is found to be insufficient, an appropriate message is output. For singly reinforced sections, the opposite moment is also checked, and if it is not zero or very small, a message indicating the fact that a section subject to moments in both directions has been designed as a singly reinforced section is output.

b) Columns. Since the actual section of the column is available, there is no need to assume a trial section. The effective length of the column and the resulting increased design quantities to take into account the strength reduction are evaluated. The column is then checked by the working stress or ultimate strength method, as specified, by evaluating the left hand side of inequality (3.65), (3.70) or (3.80), whichever is applicable, for each of the six sets of design quantities. The column is considered to be safe if all six values of the left hand sides of the applicable inequalities are less than or equal to 1.0. If any of the values exceeds 1.0, the column is deemed unsafe and a message to this effect is output.

Checking on the basis of a partial analysis has not been included in the model, because of the inherent approximations involved in the generation of the missing design quantities.

3.10 Quantity Take-Off

As mentioned in Art. 2.8, the quantity take-off procedure computes the amounts of concrete and main steel used in the structure. These quantities are accumulated story by story and the following values are output:

1. Volume of concrete and weight of steel in the slabs of each floor.

2. Volume of concrete and weight of steel in the columns of each story.

3. Total volume of concrete and weight of steel in the slabs.

4. Total volume of concrete and weight of steel in the columns.

5. Total volume of concrete and weight of steel.

The concrete volumes are evaluated in cubic yards and the steel weights in pounds.

The computation of concrete volumes is straightforward. The volume of a slab is given by the product of the corresponding bay and aisle widths and the thickness of the slab, and the volume of a column is obtained by multiplying the area of the column section by the length, which is taken to be the story height.

The volume of steel in a column is found by multiplying the area of longitudinal reinforcement in the column by the story height.

The computation of the volume of steel in a slab is performed in the following manner. The steel volume in each column and middle strip is computed separately. Considering both directions, each slab has four column strips and two middle strips. For each strip the design procedure yields the top and bottom steel areas per foot width required at the ends and middle of the strip, as shown in Fig. 3.16(a). For the purpose of the quantity take-off, the required steel is assumed to be provided by bars located as shown in Fig. 3.16(b). The lengths of the bars indicated in Fig. 3.16(b) are taken from Fig. 2104(g)a of the ACI Code, and in the case of bent bars the additional length due to the bends is ignored. The eight areas of steel corresponding to the different locations are evaluated as follows:

A_1 = Lowest value among A_{b1}, A_{b2} and A_{b3}

$A_2 = A_{t2}$

A_3 = Lowest value among $(A_{b2}-A_1)$, $(A_{t1}-A_2)$ and $(A_{t3}-A_2)$. If A_3 is negative it is set equal to zero.

$A_4 = A_{b2}-A_1-A_3$

$A_5 = A_{t1}-A_2-A_3$ (=0, if negative)

$A_6 = A_{t3}-A_2-A_3$ (=0, if negative)

$A_7 = A_{b1}-A_1$

$A_8 = A_{b3}-A_1$

Once the eight steel areas are obtained, they are multiplied by the appropriate length and strip width, and then summed to yield the steel volume for the strip. The summation of the volume of steel in the six strips of a slab yields the volume of steel, and hence the weight of steel, in the slab.

CHAPTER 4

COMPARISON OF ANALYSIS AND DESIGN PROCEDURES

In this chapter, a simple structure is used to compare the results of the analysis and design procedures described in the preceding chapters. The numerical results obtained using the analysis alternatives incorporated in the model are compared and discussed first. This is followed by a comparison of the member design quantities and of the results obtained from the design procedures.

4.1 Description of Structure

The structure used in this chapter is a symmetrical four story building with two bays and two aisles, as shown in Fig. 4.1. The figure also shows the numbering scheme for the members. The constants required for the analysis and design were specified as follows:

Compressive strength of concrete, $f'_c = 3000$ psi

Allowable concrete stress, $f_{ca} = 1350$ psi

Unit weight of concrete, $w_c = 145$ pcf

Yield strength of steel, $f_y = 40{,}000$ psi

Allowable steel stress, $f_{sa} = 20{,}000$ psi

Modular ratio, $m = 9.0$

Poisson's ratio, $\mu = 0.15$

Clear cover for slab reinforcement, $d_{tc} = d_{bc} = 0.75$ in.

Clear cover for column reinforcement, $d_{cc} = 1.5$ in.

Minimum percentage of column reinforcement, $p_t^{min} = 1\%$

Maximum percentage of column reinforcement, $p_t^{max} = 5\%$

In addition, the input side dimension, t, for all the columns was taken as 20 in., and the trial percentage for column reinforcement, p_t^{tr}, was specified to be 4%.

The loading conditions applied on the structure were as follows:

Superimposed dead load:

Floors 1 and 2: 40 psf on all panels

Floors 3 and 4: 50 psf on all panels

Live load:

Floors 1 and 2: 80 psf on all panels

Floors 3 and 4: 100 psf on all panels

Symmetrical lateral load:

Lateral loads applied at the joints of the structure as shown in Fig. 4.2.

Unsymmetrical lateral load:

Lateral loads applied as shown in Fig. 4.3.

The depths of the slabs evaluated by the empirical procedure described in Art. 3.6.1.1 were as follows:

All panels on Floors 1 and 2: d_t = 7.0 in.

All panels on Floors 3 and 4: d_t = 7.5 in.

4.2 Comparison of Analysis Alternatives

The structure was analyzed using all the analysis alternatives described in Arts. 2.2.2 and 2.2.3. In the case of the plane frame analyses, only the middle longitudinal frame was analyzed.

In the case of live loads, the stress resultants for full live load on all panels, as well as the live load maximum stress resultants, were evaluated.

The unsymmetrical lateral load was included in order to determine the effect of neglecting the in-plane twisting of the slabs and to determine whether the assumption of a rigid floor is justified. The unsymmetrical lateral load was not applied in the case of the plane frame alternatives.

Space frame alternative 10 and plane frame alternative 5 are not applicable to lateral loads, and plane frame alternative 6 is not applicable to vertical loads. Therefore, when the structure was analyzed using these alternatives, the corresponding non-applicable loading conditions were not applied.

4.2.1 Comparison of Stress Resultants

The stress resultants for girder 3, floor 1 and column 4, story 1 are compared in order to illustrate the range of values obtained from the analysis alternatives included in the model. The stress resultants for the girder are given in Table 12, and those for the top end of the column in Table 13. In the tables, moments are given in kip ft. and axial forces in kips.

In Table 12, M_A and M_B represent the left and right end moments of the girder, and M_C represents the maximum positive midspan moment. The sign convention used is the standard design convention, i.e., a positive moment causes tension in the bottom fibers of the girder.

In Table 13, P represents the axial force on the column, and M_x and M_y represent the bending moments about the X and Y axes. For the vertical loading conditions, M_x is equal to zero due to the symmetry of the structure and, therefore, is not tabulated. In the case of the plane frame alternatives, M_x is not tabulated since only the middle longitudinal plane frame was analyzed.

An examination of the values tabulated in Tables 12 and 13 leads to the following observations:

1. The greatest effect on the stress resultants is caused by neglecting the axial deformations of the columns. The effect is more pronounced in the case of vertical loads than in the case of lateral loads. The girder midspan moments and the column axial forces are affected to a lesser degree than the girder end moments and column moments, the latter being affected to the extent of 12% in some cases. This effect is somewhat surprising, since the ratio of the column axial stiffness, AE/L, to the sum of the girder transverse stiffnesses, $\sum 12EI/L^3$, is of the order of 150. It is noted that axial distortions are often neglected in practice for ratios considerably smaller than this.

2. The difference between the stress resultants due to full live load on all panels and the maximum live load stress resultants ranges between 5 and 10% in general. This is considered to be within tolerable limits for design purposes in most cases.

When the maximum live load stress resultants are computed, the minimum stress resultants at each point are also obtained. The quantities in parentheses in Table 12 represent the minimum stress resultants for space frame alternatives 1 and 9 and plane frame alternative 1. These values are also summed into the appropriate design quantities, and even though the minimum stress resultants are generally small, their effect on the design quantities may be relatively high. In particular, the maximum live load effect may sometimes call for a doubly reinforced section, whereas the corresponding full live load effect may indicate that a singly reinforced section is satisfactory.

3. Neglecting the in-plane twisting of the floors is dangerous when the structure is subject to unsymmetrical lateral loads. This is illustrated in Table 13 by the large difference between the corresponding values obtained for M_x using space frame alternatives 1 and 3. A comparison of the values obtained for M_x using space frame alternative 1 against alternatives 5, 6 8 and 9, on the other hand, shows that it is reasonable to assume that the floors behave like rigid bodies with regard to in-plane deformations.

4. The difference between the stress resultants obtained using a space frame alternative and a comparable plane frame alternative is small, being within 5% in most cases. However, this conclusion does not hold for column design quantities, as discussed in Art. 4.3.2.

5. The results obtained using plane frame alternative 6 ("shear beam" method) are grossly inaccurate. As such, this alternative is considered to be of limited value.

4.2.2 Comparison of Lateral Column Displacements

The lateral X displacements of the center column (column number 5) due to the application of the symmetrical lateral load are shown in Table 14. The values obtained for all the space and plane frame alternatives applicable to lateral loads are included in the table. The following conclusions can be drawn from the results:

1. All the space frame alternatives yield approximately equal values for the displacements. Plane frame alternatives 1-5 also yield approximately equal values. However, the displacements obtained using the plane frame alternatives are between 15 and 25% larger than those obtained from the space frame alternatives. This occurs because the restraint provided by the girders in the transverse direction is not taken

into account in a plane frame analysis.

2. As for the stress resultants, plane frame alternative 6 yields very inaccurate results. The use of this alternative is, therefore, questionable even for approximate dynamic analyses.

4.2.3 Comparison of Computer Time

The computer execution times required for the analysis of the example structure using the different alternatives are compared in Table 15. The values given for the plane frame alternatives are the times required for the analysis of the middle longitudinal frame only.

It can be seen from the table that the time required for all the plane frame alternatives is approximately 50 seconds. The time difference between the alternatives is very small, because the number of degrees of freedom is small in all cases. If larger plane frames were analyzed, the time difference between the alternatives would be more significant.

The time difference between the space frame alternatives is more marked, because of the larger range in the number of degrees of freedom. Therefore, the use of a simplified space frame alternative, such as alternative 9, is economically justifiable for a preliminary design.

A comparison between the times required for the space and plane frame alternatives suggests that it is generally not economical to use a plane frame alternative except when a partial analysis of the structure is performed.

4.3 Comparison of Member Design Methods

The example structure was designed using space frame alternatives

1 and 9 and plane frame alternative 1. Furthermore, for each of the alternatives, the structure was designed by both the working stress and ultimate strength methods. In the case of plane frame alternative 1, all six plane frames were analyzed in order to obtain a complete set of design quantities.

The loading conditions applied on the structure were the first three loading conditions given in Art. 4.1, i.e., superimposed dead load, live load, and symmetrical lateral load. The lateral load was broken up into two lateral loadings, one consisting of the lateral loads in the X direction, and the other of the loads in the Y direction. Both lateral loadings were specified to be reversible.

In the case of live loads, the maximum live load stress resultants were evaluated and used to compute the member design quantities.

The loading combinations for which the structure was designed were as follows:

a) Working Stress Method. Two combinations were specified:

1. $DL+LL$

2. $0.75(DL+LL \pm WL_1 \pm WL_2)$

where DL, LL, WL_1, and WL_2 represent the dead load, live load, and the two lateral loadings, respectively. The second loading combination reflects the practice of increasing the allowable stresses by 33-1/3% when lateral loads are included.

b) Ultimate Strength Method. As recommended in Section 1506(a) of the ACI Code, the loading combinations considered were:

1. $1.5DL+1.8LL$

2. $1.25(DL+LL \pm WL_1 \pm WL_2)$

3. $0.9DL \pm WL_1 \pm WL_2$

4.3.1 Comparison of Slab Design Quantities

The design quantities obtained for the working stress and ultimate strength design of slab 1, floor 1 are tabulated in Table 16. The table includes the quantities for all three analysis alternatives considered. The locations corresponding to the twelve positive and negative quantities are shown in Fig. 2.7(a). For the girder end moments (quantities 1-8), the positive design quantities actually represent the minimum negative moments. Similarly, for the midspan moments (quantities 9-12), the negative design quantities represent the minimum positive moment.

Since the design quantities are obtained by combining the stress resultants in accordance with the loading combination data, the quantities in Table 16 for locations 3, 4 and 10 can be derived from the corresponding stress resultants tabulated in Table 12.

An examination of Table 16 reveals that the design quantities obtained using the three alternatives are very similar. The results for space and plane frame alternatives 1 agree closely, whereas those for space frame alternatives 1 and 9 differ by 3 to 10% for the end moments, and 0 to 1% for the midspan moments.

4.3.2 Comparison of Column Design Quantities

The design quantities for column 4, story 1 are presented in Table 17. In the table, the lower end of the column is referred to as end A and the upper end as end B. The space frame analyses yield three sets of design quantities for each end of the column, whereas only two sets per end are obtained from the plane frame analysis, as described in Art. 3.4.2, the second set actually representing the maximum values of M_x and M_y and the higher value of P associated with M_x or M_y.

A comparison of the design quantities from the different alternatives shows that for the particular column the axial forces agree very closely, while the moments exhibit a greater variation. The moments obtained from the space frame alternatives compare quite closely, the discrepancies being under 10% normally. The differences between space and plane frame alternatives 1, however, are greater, and may be as high as 40-50% for some of the moments. The reason for this large difference is that a plane frame analysis does not take into account the torsional resistance of the lateral girders.

It should be noted that when a plane frame analysis is performed, each frame is considered in turn and the specified loading combinations are performed before proceeding to the next frame. When a structure is subject to lateral loads in both X and Y directions, the combined column axial forces, P, obtained from each frame analysis includes the effect of only one of the lateral loads. Therefore, since the design quantities are created by taking the higher of the two axial forces, as described in Art. 3.4.2, the axial force due to one of the two loads is not included in the design quantities. When a space frame analysis is performed, on the other hand, the computed axial forces include the effects of both lateral loads. For this reason, the discrepancy between the design quantities obtained via space and plane frame analyses may be even greater than Table 17 suggests. In particular, the discrepancies increase from the top story downwards.

4.3.3 Comparison of Slab Design

The slab depths are determined empirically and are independent of both the analysis alternative and design method. Therefore, only the required slab steel can be compared. For the purpose of illustration, the steel required in the longitudinal and lateral directions of slab 1, floor 1 is shown in Tables 18 and 19, respectively. The values given in the tables are the top and bottom areas of steel in square inches per foot width in the column and middle strips at the locations shown.

A comparison between the areas of steel obtained using the different analysis alternatives shows the same trend as that observed in the design quantities. The differences are comparatively small, and in many cases may disappear once the bar size and spacing are selected.

A comparison between the results obtained using the two design methods shows that the working stress method is consistently conservative. The steel areas computed using the ultimate strength method are generally about 15% lower than the corresponding areas provided by the working stress method. In some cases, the working stress method even calls for doubly reinforced sections at points where the ultimate strength method requires singly reinforced sections.

4.3.4 Comparison of Column Design

Since the structure and the loads are symmetrical, it is sufficient to discuss the design of columns 1, 2, 4 and 5 only. The column side dimension, t, and the total area of longitudinal reinforcement, A_{st}, required for these columns are given in Table 20 for the three analysis alternatives and two design methods considered.

A comparison between the column sizes obtained using the different alternatives shows that the two space frame alternatives yield almost identical results. When plane frame alternative 1 is employed, some columns tend to be slightly overdesigned because of the higher moments in the design quantities, as pointed out in Art. 4.3.2.

As in the case of slab steel, the working stress method is found to be consistently conservative in comparison with the ultimate strength method. The difference between the two design methods is heightened by the fact that the formula recommended by the ACI Code, and used in the model for the design of tension-controlled columns by the working stress method (Eq. 3.70), is definitely overconservative in comparison with Bressler's formula (Eq. 3.75) which is used in this study for the design of tension-controlled columns by the ultimate strength method.

4.3.5 Comparison of Concrete and Steel Quantities

The total volumes of concrete and weights of steel required for the slabs and columns of the example structure are given in Table 21 for the three analysis alternatives and two design methods considered. The concrete volumes are computed in cubic yards and the steel weights in pounds.

The differences between the analysis alternatives and design methods noted above are also reflected in the quantities shown in the table. The volume of concrete for the slabs is, of course, the same in all cases. The other quantities show that the choice of analysis alternative does not affect significantly the total quantities. On the other hand, the quantities obtained using the ultimate strength method are between 20 and 25% lower than the corresponding quantities obtained using the working stress method.

The comparisons given in this chapter are based upon a single cycle of analysis and design, rather than on a converged final design. However, the trends discussed in this chapter also apply to converged designs, examples of which are presented in Chapter 6.

CHAPTER 5

DESCRIPTION OF SYSTEM

The model described in Chapters 2 and 3 is incorporated into a system for the analysis, design, and checking of flat plate reinforced concrete buildings. The system is conceived for use in an on-line environment, but, due to the inaccessibility of appropriate facilities, the present version of the system was developed using the conventional batch mode. However, only a few minor changes are required to effect a conversion from the present system to an on-line system. The objective, organization and capabilities of the system are described in the following articles.

5.1 Objective of System

The objective is to develop an integrated man-machine system for the analysis, design, and checking of flat plate reinforced concrete buildings. It is not the objective of this study to automate the process completely, i.e., it is not expected that the designer will feed a deck of data cards into the computer and receive, without further ado, the member sizes, bar schedules, etc., as output. The purpose, on the other hand, is to create an environment of close cooperation between the engineer and the computer, so that the engineer is able to fulfill his functions more effectively by relegating to the computer the computational and data-processing tasks associated with the design.

5.2 Organization of Computer Program

The organization of the present off-line computer program is described below. The program is organized in such a way that the various phases can be accessed easily by a designer or a checker. Since the complete program exceeds the memory capacity of the computer used (IBM 7094), the program is divided into several segments called links. The program segments are stored in secondary storage, and they are brought into the primary memory whenever required during the processing.

The program is divided into seven links as follows:

Link	Functions	Links Called
1	Input of data and communication with other links	2,5,6,7
2	Generation of stiffness matrix and forward pass of solution	3
3	Backward pass of solution	4
4	Loading Combinations and Design Quantities	1
5	Design of Members	1
6	Quantity take-off	1
7	Checking of members	1

Link 1 acts as the master link, and is used to read in all the input data, as well as to serve as the communication link with the other links.

A block diagram showing the organization of the program is given in Fig. 5.1. A numeric flag is used to specify the next task to be performed. The flag takes on the values 1, 2, 3, and 4 which are interpreted by the program as follows:

Flag	Interpretation
1	A new structure is being started.
2	A modification has to be made in the problem being processed.
3	Design or check the structure, depending upon the value of the function flag.
4	Compute the material quantities for the structure.

A second numeric flag is used to indicate the function to be performed, i.e., to design a structure or to check a previously designed structure.

The input data pertaining to the structure are divided into several logical data blocks. Each block is preceded by an alphabetic control statement such as GEOMETRY, GROUPS, etc., which is followed by the corresponding data block.[5] Upon identifying a control statement, the program executes the appropriate subroutine which reads in the data block associated with that control statement.

When the system is used for analysis and design, the input data must consist of the following control statements and the associated data:

GEOMETRY

GROUPS

CONSTANTS

ANALYSIS DATA

LOADINGS

LOAD COMBINATIONS

After the control statements and associated data are read in, the control statement SOLVE is used to initiate the processing. After the structure has been designed once, the control statement REPLACE can be used to replace the trial member properties used for the previous cycle by the new member properties obtained.

When the system is used for the purpose of checking, the following data blocks received from the designer must be input:

1. Geometry of the structure.
2. Slab depths and reinforcement details.
3. Concrete and steel areas of the columns.

The remaining data are supplied by the checker, and must consist of the following control statements and the associated data:

CONSTANTS

ANALYSIS DATA

LOADINGS

LOAD COMBINATIONS

Finally, as in the case of analysis and design, the statement SOLVE is used to initiate the solution.

The format for the input of data associated with the control statements is patterned after a problem-oriented language. However, in the present version, code numbers are used in place of words. A detailed description of the input variables and the present form of input are given in Appendix B.

5.3 Applications of System

The system can be applied to the analysis and design of a structure, as well as to the checking of a previously designed structure, as described below.

5.3.1 Analysis and Design

A block diagram demonstrating a possible man-machine system for the analysis and design of a structure is shown in Fig. 5.2. In the figure, solid boxes represent the tasks performed by the program while the dotted boxes represent the decisions and actions of the designer. In the present off-line version of the system, the sequence of the designer's actions is specified in advance, while some of the decisions are made by the program based upon calculated numerical values.

The system is organized in such a manner that when the designer has arrived at a final design, he can output the geometry and the member sizes and reinforcement details on punched cards or magnetic tape. This output data can then be transmitted to the checker, who can use it directly as input during the checking phase.

The various possible analysis and design applications of the system are as follows.

5.3.1.1 Single Cycle Design

The system can be used to obtain a preliminary design of a structure in a single cycle. The input data pertaining to the structure are specified as described in Art. 5.2, and the statement SOLVE is used to initiate the solution.

After the structure is analyzed and the design quantities are evaluated, the program returns to Link 1. At this point, the designer may choose to make modifications in the original data and reanalyze the structure. The same statements used to control the input of the original data are used for the input of the modified data. A drawback of the present system, however, is that when any data in a particular data block is modified, the whole data block must be input again.

If the designer does not desire to make any modifications in the data after the design quantities have been evaluated, he may proceed to the design of the members of the structure. After the design process is complete, the program returns to Link 1, and the designer may now request that the material quantities be computed.

The above process constitutes a single cycle design, and was employed to obtain the results reported in Chapter 4.

5.3.1.2 Iterative Analysis and Design

The system can be used to iterate upon the analysis and design of a structure. The designer can request a specific number of iterations, or continue the iteration process until the design converges. In the present version of the program, the design is considered to have converged when none of the column sizes changes during an iteration. The slab depths do not enter into the convergence criterion, because they are determined empirically at the beginning of the design and, therefore, do not change from cycle to cycle.

After the first cycle of analysis and design is complete, the designer can use the REPLACE statement to replace the trial member properties with the new calculated properties. He can then proceed to reanalyze and redesign the structure using the SOLVE statement.

In addition to replacing the old member properties after each cycle of iteration, the designer can also modify any of the original input data for the problem. For instance, the design may be started using space frame alternative 9 for the analysis, and after one or more cycles of iteration the alternative may be changed to the more accurate alternative 1.

5.3.1.3 Iterative Single Cycle Designs

When the analysis and design of a structure is being iterated upon to convergence, it is mandatory to REPLACE the old member properties before performing another cycle.

On the other hand, by not using the REPLACE statement between iteration cycles, the designer can effectively perform iterative single cycle designs. For instance, the designer may wish to study the effect of a particular parameter on the design of the structure. To do this, he can design the structure once, modify the parameter in question, and redesign the structure without replacing the old member properties. The process may be continued to obtain as many comparative designs as desired.

5.3.2 Checking

A block diagram of the possible application of the system to checking is given in Fig. 5.3. A checking agency, which may be the design office itself or a completely separate organization, can use the system to determine whether or not the designed structure meets the provisions and requirements stipulated by the Building Code or other authority. It is quite conceivable that the design criteria and checks employed by the checking agency may be different from those used by the design office and, therefore, it is possible that a design found acceptable in the design phase may result in non-compliance during the checking phase, and be returned to the designer for revision.

Checking is essentially a single cycle process. The program is initiated by reading in the data supplied by the designer. This is followed by the data provided by the checker, input under control of the appropriate control statements. Finally, the SOLVE statement is used to begin the checking procedure. After the design quantities are evaluated, the program returns to Link 1. If the checker now wishes to make any modifications in the data supplied by himself, he can do so and reanalyze the structure. If no modifications are desired, the program transfers to Link 7, which checks the design of the members.

When the checking procedure is complete, the program again returns to Link 1. If the checker desires to make any alternate checks, he can modify the appropriate data and recheck the structure. This sequence of operations can be continued until the checker has accumulated sufficient information to decide whether to accept or reject the design.

CHAPTER 6

EXAMPLES OF APPLICATION

In this chapter, several examples are presented to illustrate the applications of the system described in Chapter 5, and to study the effect of using some of the facilities incorporated in the system. The effects due to the initial trial sizes of the columns, the use of member groups, the use of different analysis alternatives, and the application of partial analyses are studied. The application of the system to the checking of a structure is also illustrated.

6.1 Description of Structure

The structure used for the examples in this chapter is a three story building with three bays and two aisles, as shown in Fig. 6.1. The data associated with the CONSTANTS statement are the same as given in Art. 4.1, except that p_t^{max} was assumed equal to 6%. The trial side dimensions for the columns were input separately for each of the examples and are given below in conjunction with the examples.

The loading conditions applied on the structure are the same for all the examples, and are as follows:

Superimposed dead load:

Floor 1: 40 psf on all panels

Floor 2: 50 psf on all panels

Floor 3: 60 psf on all panels

Live load:

Floors 1 and 2: 100 psf on all panels

Floor 3: 120 psf on all panels

Lateral load in X direction:

Lateral joint loads applied as shown in Fig. 6.2.

Lateral load in Y direction:

Lateral joint loads applied as shown in Fig. 6.3.

For the live load analysis, full live load on all panels was used instead of the maximum live load effect.

In all the examples, the structure was designed using the working stress method, and the following commonly used loading combinations were specified for the design:

1. DL+LL
2. $0.75(DL+LL \pm WL_1 \pm WL_2)$

where DL represents the dead load, LL represents the live load, and WL_1 and WL_2 represent the two lateral loadings.

6.2 Example 1 - Effect of Trial Column Sizes

Because of the non-linear nature of the iterative design process, the final column sizes cannot be predicted from the initial values assumed. In order to study the effect of the trial sizes assumed for the columns, the structure was designed three times with different initial trial sizes. A trial size as close as possible to the expected final size was selected for all the columns in the first design. For the second design an underestimate of the final size was used, and for the third design an overestimate was used, as follows:

DESIGN NO.	TRIAL COLUMN SIZE, t
1	21 in.
2	12 in.
3	30 in.

In each case, space frame alternative 9 was used for the analysis, and the analysis and design process was iterated upon upto convergence using the REPLACE statement.

With the initial t of 21 inches, five cycles of analysis and design were required for convergence, whereas six cycles were required with the initial t of 12 inches and seven cycles with the initial t of 30 inches. Therefore, it may be concluded that the initial trial size does not affect significantly the rate of convergence.

The results of the design of the columns for all three cases are given in Table 22. It can be seen from the table that the design process did not converge to the same final sizes and steel areas for the three designs. This, of course, is not unexpected, since an indeterminate structure has many satisfactory solutions. Taking design no. 1 as the "standard", a comparison of designs 1 and 2 shows that the final sizes of as many as half of the columns are different. A comparison of designs 1 and 3 shows fewer changes, probably because the trial size used for design no. 1 also proved to be a slight overestimate for many of the columns. It should be noted, however, that for four of the columns, design no. 3 yields smaller column sizes than design no. 1.

The empirically evaluated slab depths are as follows for all three designs:

Slabs	Floors 1,2	Floor 3
1,3	8.5	9.0
2	9.0	10.0
4,5,6	10.0	11.0

The final slab steel areas obtained from the three designs are compared for slab 4, floor 2 in Table 23. The table shows that there is not much difference between the three designs. Moreover, if actual bar sizes are substituted for the steel areas, the differences would be further diminished.

The material quantities required for the three designs are shown in Table 24. In the table, the quantities required after one cycle of analysis and design, as well as the final quantities for the converged designs are given. A comparison of the quantities for the three final designs shows that the initial trial column size of 12 inches (design no. 2) results in the lightest structure, apparently due to the fact that the slabs are used more efficiently. This observation is supported by the fact that the slab steel weight is the highest for design no. 2.

The results suggest that it may be desirable to start the analysis and design process using a slightly underestimated trial size for the columns. However, this is only recommended if the design is carried to convergence. If the design process is stopped before convergence, an underestimated trial size is liable to result in an unsafe design. If the designer does not plan to continue the design process until convergence, he should start with slightly overestimated trial sizes.

Since there are many possible solutions for a structure, the design process should ideally be converted into an optimization procedure. In buildings of the type considered in this study, optimum cost is generally not achieved by designing a structure of minimum weight, but by keeping the cost of formwork and labor down to a minimum. This goal may be achieved to some extent by specifying appropriate member groups. From an aesthetic standpoint, the specification of member groups is also desirable to avoid obtaining larger columns in the upper stories than in the lower ones, as obtained in all three of the designs compared above. In view of the above comments, it is apparent that the comparisons presented in this article are somewhat unrealistic, because each individual member was designed separately.

6.3 Example 2 - Effect of Member Groups

In order to illustrate the effect of specifying member groups upon the design of the structure, three different sets of member groups were selected, and the structure was designed using these in turn. In all three cases, the structure was analyzed using space frame alternative 9. For reference purposes, the three designs will be referred to as design nos. 4, 5 and 6.

For design no. 4, all the slabs in each story were specified to be of equal depth, and the corner, edge and interior columns of each story were placed in separate groups to form a total of nine column groups.

For design no. 5, the slabs in each of the two aisles were specified to be of equal depth in all three stories. Each column was also specified to be of the same size in all three stories, and the corner, edge and interior columns were placed in separate groups to form a total of three column groups. The slab and column groups for this design were chosen such that the same formwork could be used for all three stories in the structure.

For design no. 6, the same slab and column groups specified for design no. 5 were used. In addition, however, three program-selected groups were specified for the columns with the following lower and upper limits for the concrete area:

	Lower Limit	Upper Limit
GROUP NO. 1	300 sq. in.	370 sq. in.
GROUP NO. 2	390 sq. in.	450 sq. in.
GROUP NO. 3	500 sq. in.	580 sq. in.

The slab groups for the three designs and the resulting slab depths are given in Table 25.

The column groups, the prescribed trial sizes for each group, and the results of the first and last cycles of analysis and design are given in Table 26 for design no. 4. As described in Art. 3.7.4, within each cycle all the columns are first designed independently. Each column is then revised making the side dimension equal to the control dimension for the group, followed by an attempt to reduce the provided steel area. Both the calculated and revised results of the design are shown in the table. In the table, the control column of a group is identified by an asterisk, and it can be seen that when a column size is increased to conform to the group size, the required steel is generally reduced. Similar information is given in Table 27 for design no. 5.

The results of designs 4, 5 and 6 for all the columns are summarized and compared in Table 28. The table shows that in design no. 4 some of the middle story columns are smaller than those of the lower and upper stories, as in the case of designs 1, 2 and 3. This can be avoided by a judicious selection of column member groups, as demonstrated by the results of design no. 5. A comparison of designs 5 and 6 shows that all the columns of side dimension 20 inches in design no. 5 fell into program-selected group no. 2 and were made equal to 21 inches in design no. 6. Some of the steel areas were also increased, reflecting the fact that a stiffer member attracts larger moments.

It is noteworthy that design no. 4 required 3 cycles for convergence, and design nos. 5 and 6 required only 2 cycles. In comparison, design no. 1, for which no groups were specified, required 5 cycles to converge. This demonstrates the fact that the specification of member groups reduces the number of cycles required for convergence, due to the fact that member groups reduce the number of design variables which can change from cycle to cycle.

The material quantities obtained for designs 4, 5 and 6 are compared with each other and with the quantities for design no. 1 in Table 29. The table shows that when member groups are specified, the volume of concrete is relatively higher than for design no. 1, but the weight of steel is lower. It is, therefore, probable that the costs of materials for the designs with and without member groups are comparable.

6.4 Example 3 - Effect of Analysis Alternative

In order to study the effect of the differences in design quantities produced by different analysis alternatives upon the rate of convergence and the final design of a structure, design no. 5, in which space frame alternative 9 was used, was repeated using space frame alternative 1.

It was found that the design converged in two cycles as in the case of the design performed using analysis alternative 9. Moreover, the final member sizes were exactly the same. Minor differences were detected only in the slab steel areas and in some of the column steel areas. The total weight of steel required was only 180 lbs less than that required using alternative 9.

On the basis of the results obtained, it can be concluded that the design of a structure is generally not greatly affected by the analysis alternative employed. However, it should be noted that if fewer member groups than specified for design no. 5 are used and, therefore, a larger number of design variables are involved, the differences between the results obtained using different alternatives could be greater than in the present example. In the light of the comparisons reported in Chapter 4, moreover, it should be observed that care must be exercised in choosing an analysis alternative that is appropriate for the problem at hand.

6.5 Example 4 - Effect of Partial Analysis

Two additional designs were performed in order to examine the applicability of partial analysis to the design of a structure. The two partial analysis designs will be referred to as designs 7 and 8. The plane frames analyzed for the two designs are indicated by heavy lines in Fig. 6.4: of the seven plane frames in the structure, four were analyzed for design no. 7, and only two for design no. 8. Plane frame alternative 1 was used for the analysis of the frames. A single cycle of analysis and design was performed in each case because, presumably, only a preliminary design was being sought.

The slab groups specified and the resulting slab depths for design nos. 7 and 8 are shown in Table 30. The same groups of slabs of equal depth were specified as for design no. 5. However, within these groups the slabs for which fewer design quantities were calculated were set identical to slabs with more design quantities available. The latter are automatically selected as the control slabs for the computation of the reinforcing steel requirements of the corresponding slab groups.

For design no. 7, the same groups of columns of equal dimensions were specified as for design no. 5. Within each such group, however, the columns in the same story were specified to be identical. From each group of identical columns, the program selects one or more control columns, as described in Art. 3.8.2, and designs them independently of each other, after which the columns are revised to satisfy the member group data. The results of the design of the columns for design no. 7 are given in Table 31.

For design no. 8, only groups of identical columns were specified, with every column being identical throughout its length. This was done because it is not warranted to indulge in the sophistication of refining the column steel areas when only two plane frames are analyzed. The results of the design of the columns for design no. 8 are given in Table 32. The table includes both the calculated values of t and A_{st} for the control columns, as well as the revised values for each group of columns.

The material quantities required for designs 7 and 8 are compared with each other, as well as with design no. 5 in Table 33. The concrete volumes obtained for the three designs compare favorably. However, the steel weights obtained for design nos. 7 and 8 are greater than for design no. 5 by 16% and 57%, respectively. In particular, the weights of slab steel obtained for design nos. 7 and 8 are high because the design quantities evaluated for interior frames are used for exterior frames in many cases. Note that, even if the upper and middle story column steel areas had been refined in design no. 8, the discrepancy in the total steel weight would still be very high. As expected, therefore, it can be concluded that much more realistic results are obtained by analyzing four plane frames than by analyzing two.

6.6 Example 5 - Checking of the Structure

In order to illustrate the checking capability of the system, design no. 5 was checked under the following four conditions:

i. No change in the data used for the design of the structure.

ii. A change in the allowable stress of concrete f_{ca} from 1350 to 1305 psi, corresponding to a 3-1/3% reduction.

iii. Specification of the maximum live load effect instead of the full live load effect.

iv. Conditions (ii) and (iii) simultaneously.

In addition, the structure was checked using the design output from both the first (called preliminary) and second (final) cycles of analysis and design.

The results of the checking process are tabulated in Table 34. The table shows that even when no changes in the design data are made, the concrete in the slabs is overstressed at 8 out of 324 points (18 per slab) when the preliminary output is used. In comparison, the design is found to be completely safe when the final output is used with no changes in data. This emphasizes the danger of accepting a design before convergence has been attained, especially when the initial trial column sizes are not close to the final converged values. For instance, if the first cycle output from design no. 3 (trial t = 30 inches) had been checked, the severity of non-compliance would undoubtedly be greater than obtained using the first cycle output of design no. 5.

When the maximum live load effect is considered instead of the full liveload used for the design, it is found that the steel is overstressed at nearly all the points in the slabs. Four of the 36 columns are also found to be unsafe by a small percentage. The average overstress in the slab steel is about 10%, but in isolated instances the overstress percentage is as high as 31.8% with the preliminary output and 67.6% with the final output. A contributing factor to the large discrepancies is the fact that in this design, slab groups of equal depth only were specified, so that each individual slab was designed for at least 95% efficiency. Consequently, the slab designs were extremely susceptible to any changes in the data.

CHAPTER 7

CONCLUSIONS AND SUGGESTIONS FOR FURTHER DEVELOPMENT

7.1 Conclusions

A prototype computer-aided system for the analysis, design and checking of flat plate reinforced concrete buildings has been presented.

The actual structure was idealized as an assemblage of girders and columns in accordance with the recommendations of the ACI Code and current practice. While this assumption provides good agreement between space and plane frame analyses, the distribution of panel loads to the girders appears to be somewhat conservative. However, no reasonable substitute could be implemented without resorting to a general plate analysis.

It has been shown that a large number of space and plane frame analysis alternatives, corresponding to a wide range of structural behavior assumptions, can be efficiently treated by a general procedure. However, on the basis of the numerical results obtained, it is questionable whether it is necessary to include in an operational system as many analysis alternatives as were implemented in this study.

In this study no reduction was made in live load effect for lower stories, because the procedure used to evaluate the maximum live load effects properly accumulates all the contributing quantities. On the other hand, the very large number of additional loading conditions necessary to accomplish this may prove to be uneconomical for large structures. In the examples considered in this study, the difference in the stress resultants due to full live load on all panels and the maximum live load stress resultants was only of the order of 5%.

The evaluation of the member design quantities presented a very complex logical task and required the development of an elaborate algorithm. This was especially true for the column design quantities, for which it is necessary to evaluate critical combinations of axial force and moments. The criteria employed to accumulate the column design quantities are not completely general, because it was assumed that minimum axial force would never govern. The facility for treating reversible loadings, while further complicating the computation of column design quantities, can considerably reduce the number of loadings and loading combinations that have to be considered.

The provision for specifying member groups is a valuable tool towards achieving a good design, which is both economical and aesthetically pleasing.

In the present design algorithm, an attempt is made to design the members for at least 95% efficiency. For columns, where discrete steel areas are considered, this efficiency is rarely achieved. However, this efficiency is always attained for slabs, where a continuous variation of available steel areas is assumed. It appears, from the checking example presented in Chapter 6, that the resulting slab design may be too "tight". It would be a simple matter to reduce the efficiency factor in the program or even to make it an input variable.

Because of the non-linear nature of the iterative analysis and design process, the final sizes of the members cannot be predicted from the assumed initial values. In the examples studied, it was found that more efficient designs are achieved by starting the design process with an underestimate of the final expected size for the columns.

However, an unsafe design can result if the iteration is not carried to convergence.

Even though only a conventional batch mode was available, most of the features of an on-line man-machine system could be satisfactorily simulated. It therefore appears that a system in which the designer can retain control throughout the entire design process is entirely feasible. The system described herein consequently seems to be an attractive alternative to the more highly automated design system described by Hill.[6]

7.2 Suggestions for Further Development

The present study was of necessity restricted to a single highly idealized type of structure, and can only be used in a batch mode environment. Further developments and extensions of the system should be directed along three broad paths discussed below.

a) Idealization of Structure. Improvements in the idealization of the structure should be aimed towards:

1. Generalizing the geometry and make-up of the structure by extending the model to structures of irregular geometry, columns of other shapes and reinforcement details, etc.
2. Replacing the present slab idealization for the evaluation of girder stiffnesses and uniform load distribution by a more realistic one.
3. Extending the model to two-way slab design by the addition of the required subroutines for two-way slab load distribution and the design of the supporting girders.

b) <u>Procedural Improvements</u>. Modifications and changes in the procedures incorporated in the program should include:

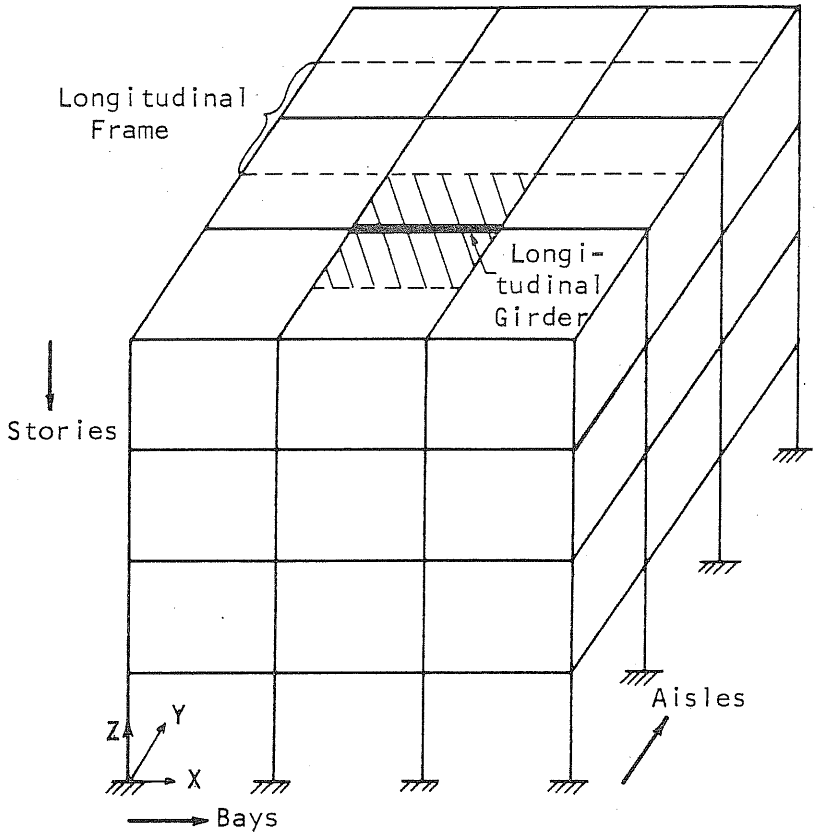
1. Generalizing some of the algorithms implemented, notably the accumulation of column design quantities and the iteration on member design and checking.
2. Replacing the tri-diagonal procedure by rewriting the solution procedure in the POST language,[7] which uses secondary storage in a more flexible and efficient fashion.
3. Extending the types of member groups included, notably by allowing individual slab strips to be treated in groups. This would permit the design of "identical" slab groups in which the slabs are mirror images of each other with regard to reinforcement.
4. Including the selection of secondary steel for slabs and columns in the design process, and the summing of the corresponding additional steel weight in the material quantities.
5. Incorporating plastic frame analysis as an alternative in the analysis algorithm.

c) <u>System Improvements</u>. The following system improvements should be given consideration:

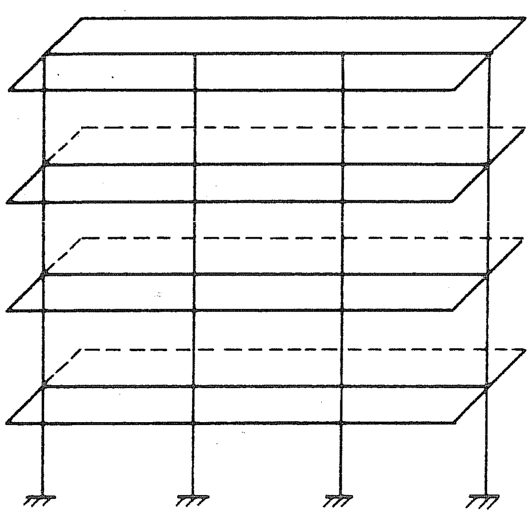
1. Converting all of the input data to a completely problem-oriented format.
2. Implementing the system in an on-line environment when the appropriate facilities become available.

3. Providing facilities to the designer for the optimization of selected design variables.

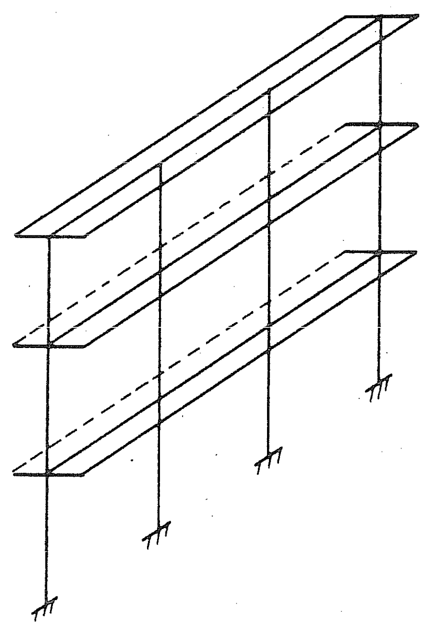
Beyond the above suggestions additional extensions and changes will undoubtedly suggest themselves as the system is applied in practice.



(a) Space Frame Idealization

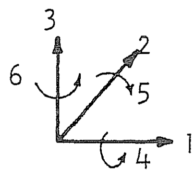


(b) Typical Longitudinal Frame

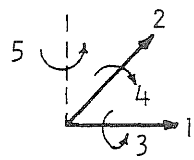


(c) Typical Lateral Frame

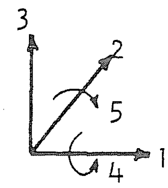
FIG. 2.1 SPACE AND PLANE FRAME IDEALIZATIONS OF STRUCTURE



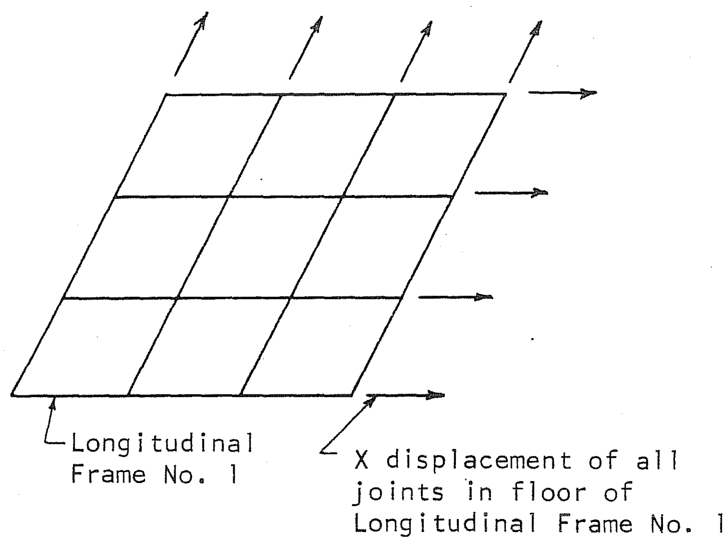
(a) Alternative 1



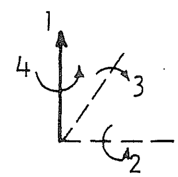
(b) Alternative 2



(c) Alternative 3

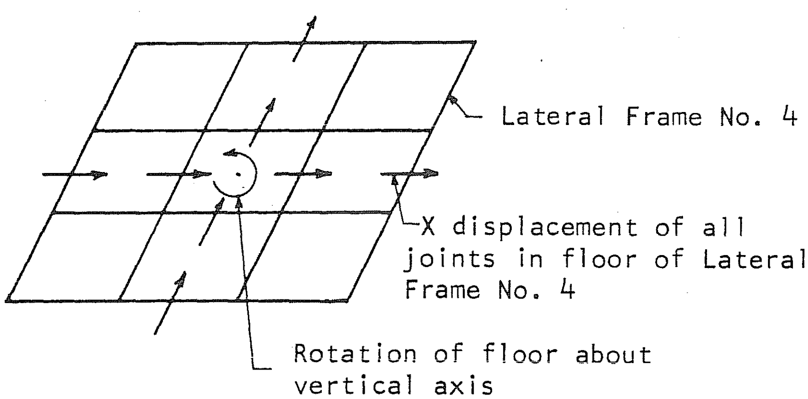


(i) Lateral Degrees of Freedom of a Typical Floor

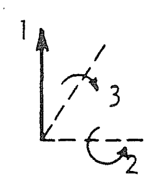


(ii) Independent Joint Displacements

(d) Alternative 4



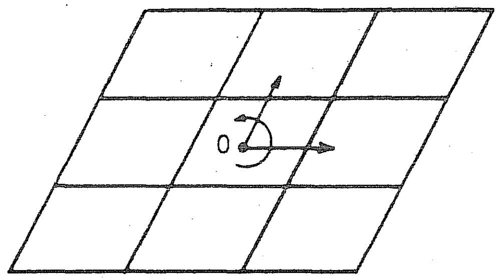
(i) In-plane Degrees of Freedom of a Typical Floor



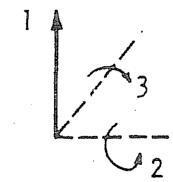
(ii) Independent Joint Displacements

(e) Alternative 5

FIG. 2.2 DEGREES OF FREEDOM FOR SPACE FRAME ALTERNATIVES

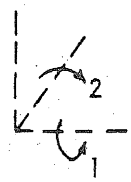


(i) In-plane degrees of freedom of a typical floor



(ii) Independent Joint displacements

(f) Alternative 6



(g) Alternative 10

FIG. 2.2 (CONTINUED)

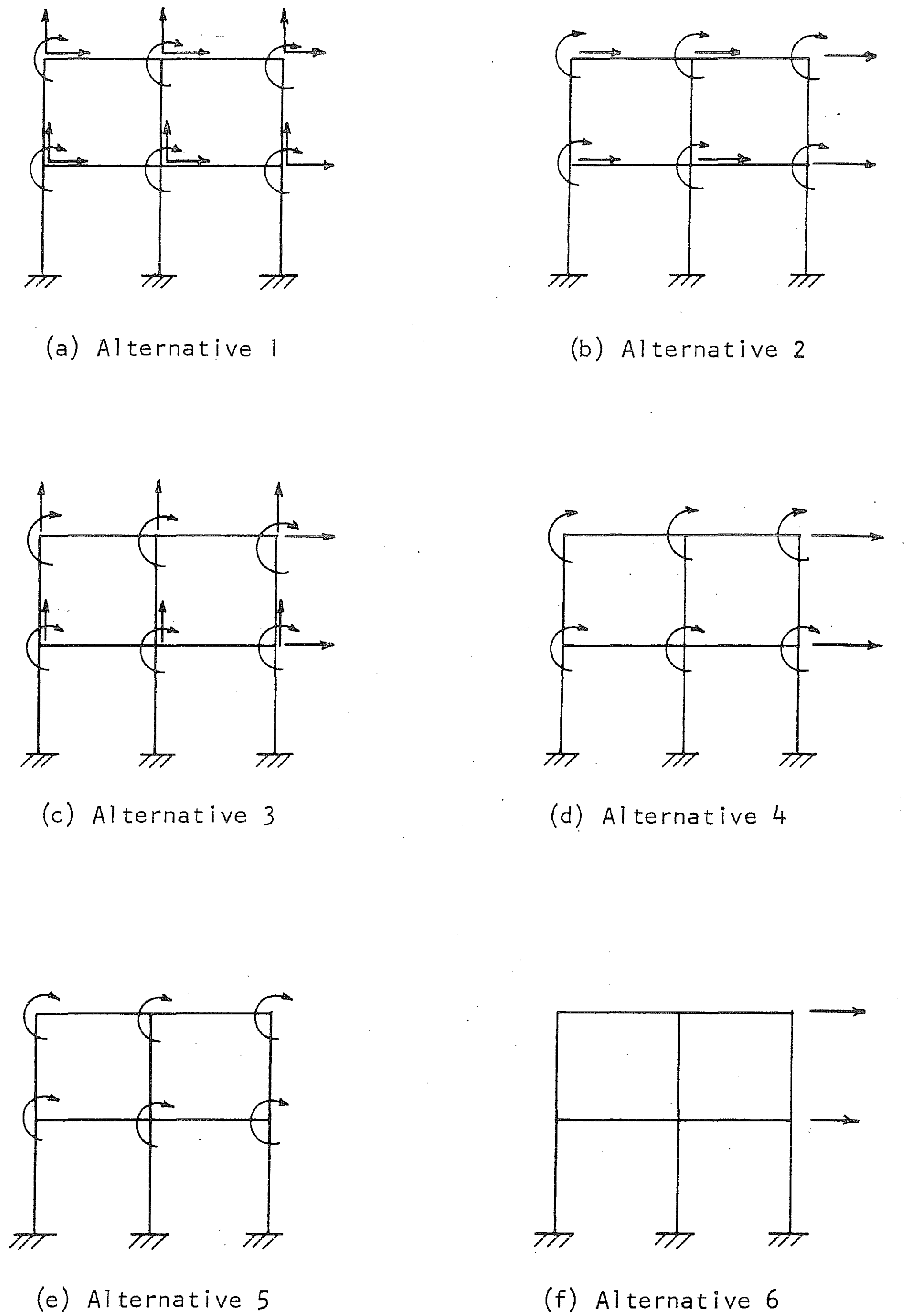


FIG. 2.3 DEGREES OF FREEDOM FOR PLANE FRAME ALTERNATIVES

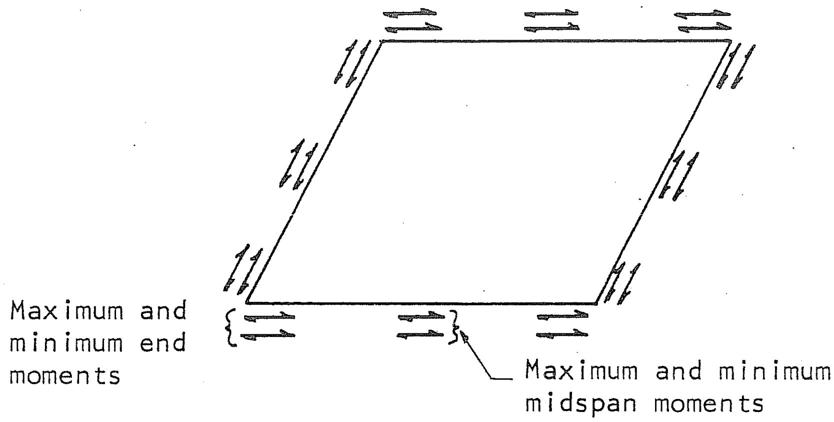


FIG. 2.4 DESIGN QUANTITIES OF A SLAB

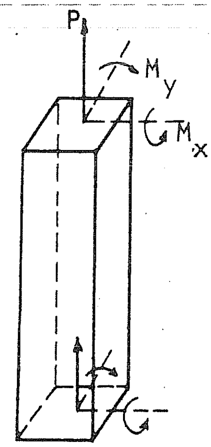


FIG. 2.5 DESIGN QUANTITIES OF A COLUMN

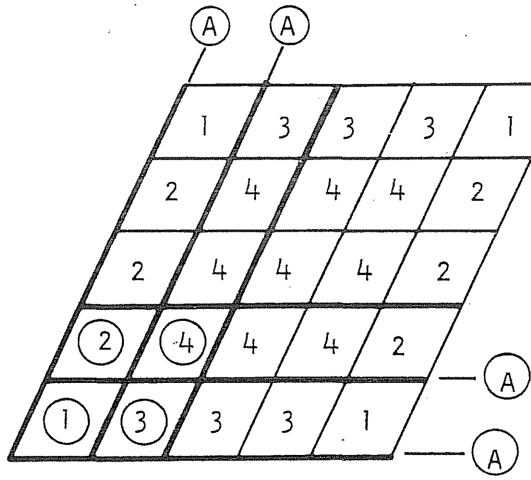


FIG. 2.6 TYPICAL FLOOR SHOWING SLAB GROUPS

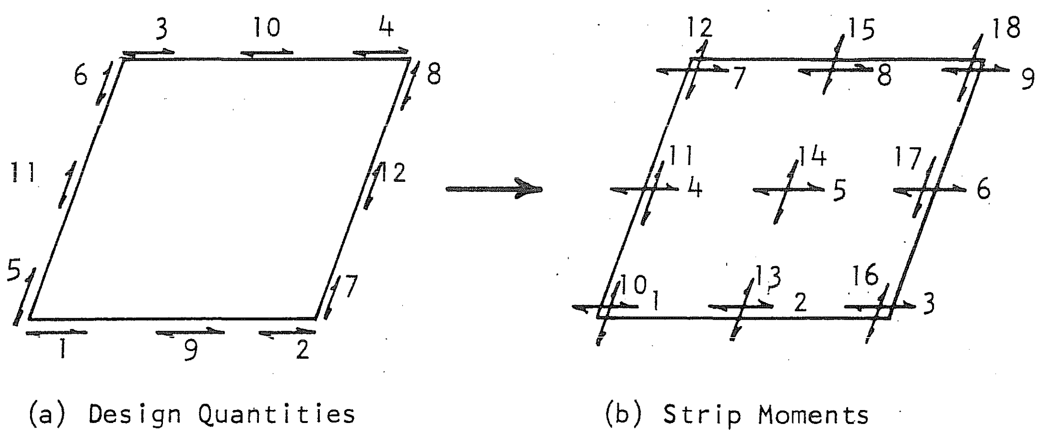


FIG. 2.7 CONVERSION OF DESIGN QUANTITIES INTO STRIP MOMENTS

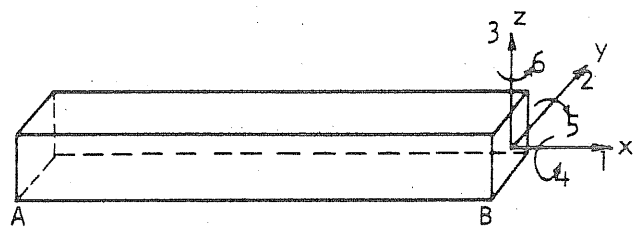


FIG. 3.1 END DEFORMATIONS OF A SPACE FRAME MEMBER

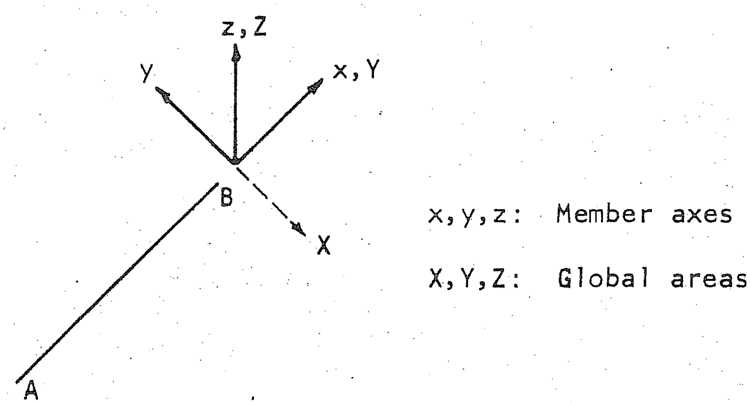


FIG. 3.2 RELATION BETWEEN MEMBER AXES AND GLOBAL AXES FOR A LATERAL GIRDER

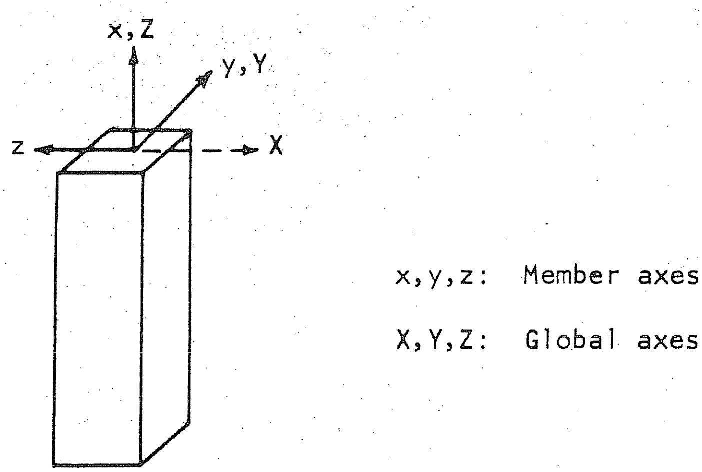


FIG. 3.3 RELATION BETWEEN MEMBER AXES AND GLOBAL AXES FOR A COLUMN

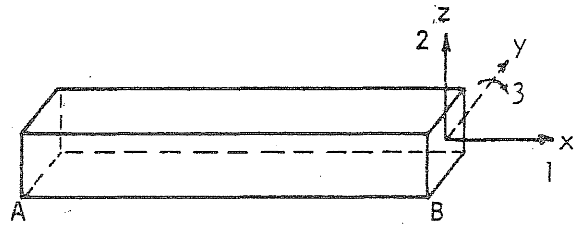


FIG. 3.4 END DEFORMATIONS OF A PLANE FRAME MEMBER

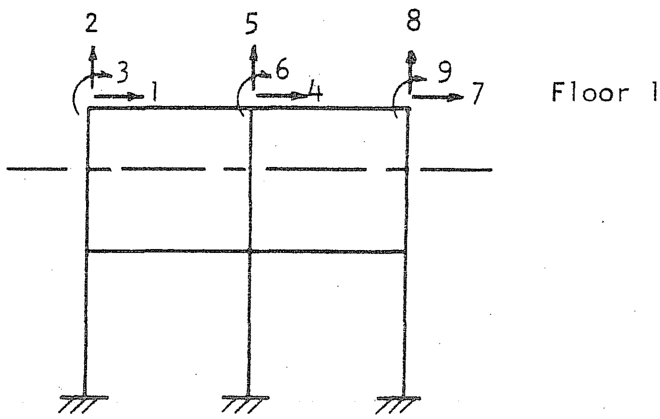


FIG. 3.5 JOINT DISPLACEMENTS FOR ALTERNATIVE 1

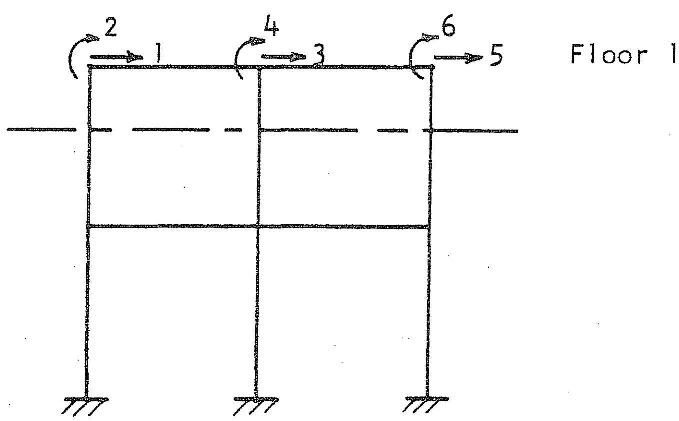


FIG. 3.6 JOINT DISPLACEMENTS FOR ALTERNATIVE 2

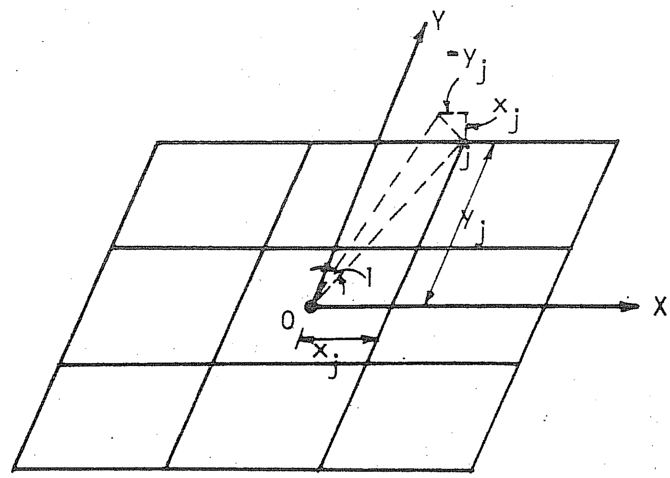


FIG. 3.7 DISPLACEMENTS OF JOINT J DUE TO A RIGID BODY MOTION OF THE FLOOR

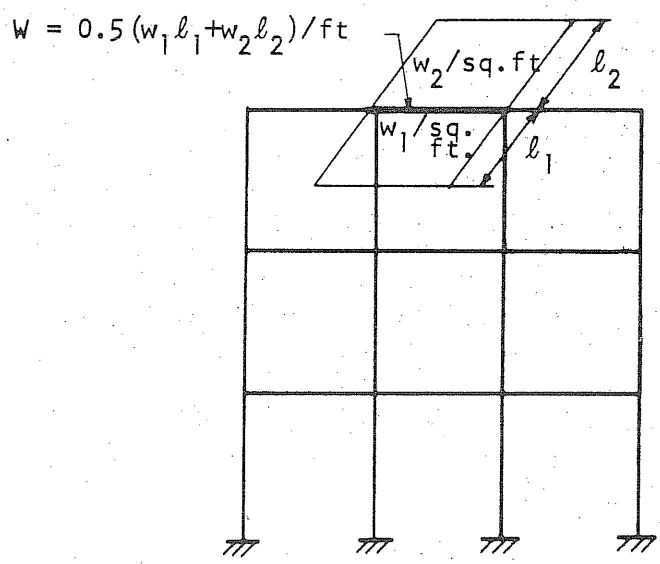
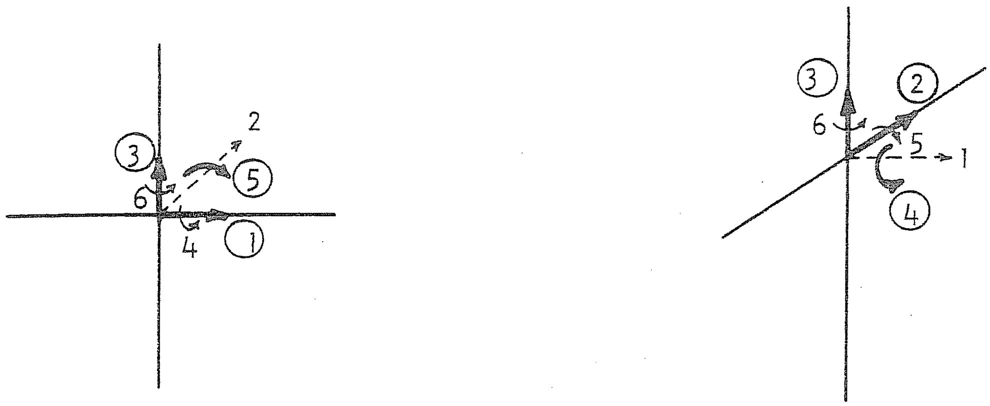
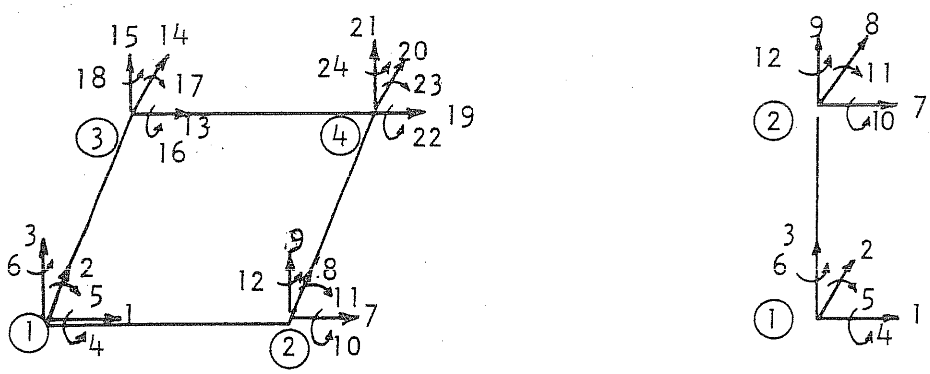


FIG. 3.8 EQUIVALENT UNIFORM LOAD ON A GIRDER



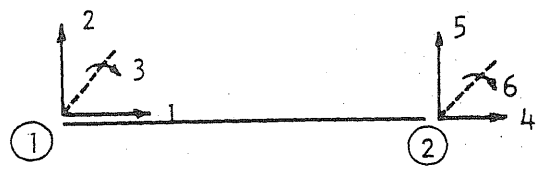
(a) Longitudinal Frame (b) Lateral Frame

FIG. 3.9 GENERALIZED DISPLACEMENTS OF PLANE FRAME JOINTS

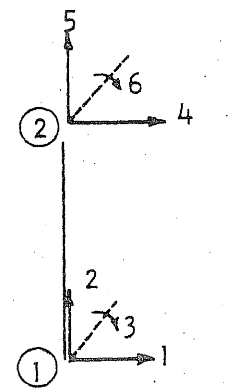


(a) Slabs (b) Columns

FIG. 3.10 NUMBERING OF MEMBER END DEFORMATIONS FOR SLABS AND SPACE FRAME COLUMNS

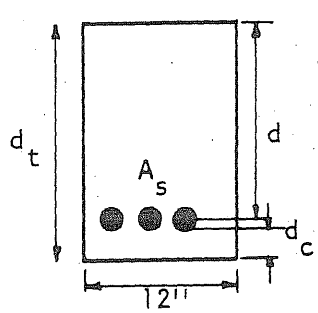


(a) Girders

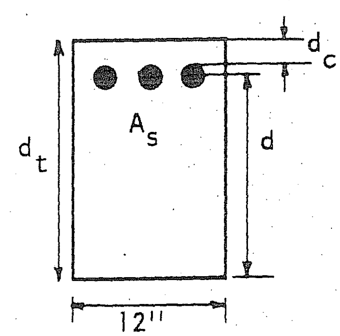


(b) Columns

FIG. 3.11 NUMBERING OF MEMBER END DEFORMATIONS FOR PLANE FRAME GIRDERS AND COLUMNS



(a) Positive Moment



(b) Negative Moment

FIG. 3.12 SINGLY REINFORCED SECTION

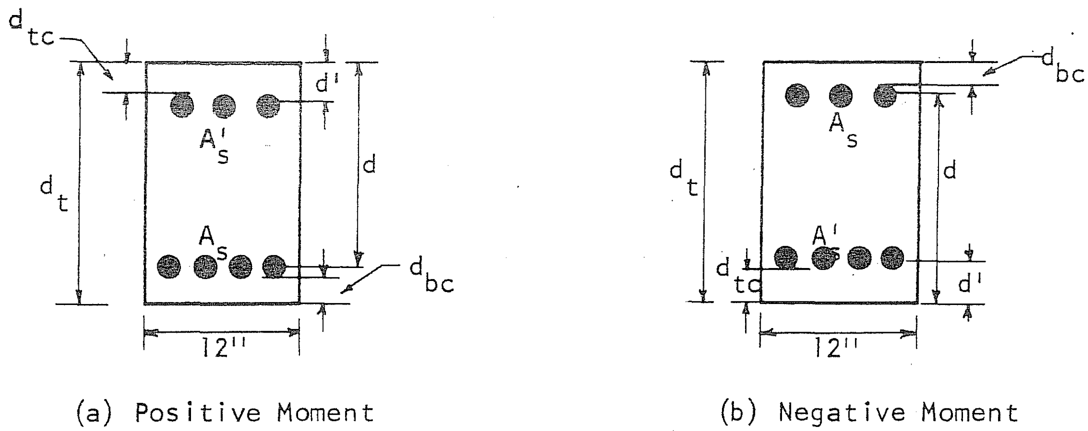


FIG. 3.13 DOUBLY REINFORCED SECTION

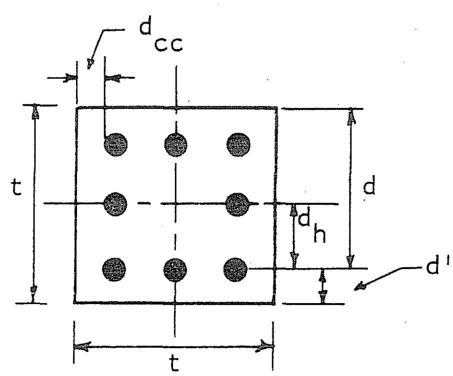


FIG. 3.14 COLUMN SECTION

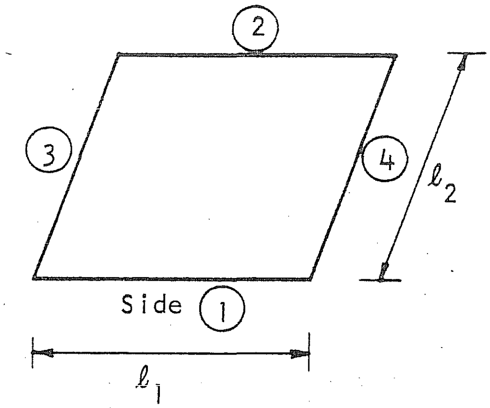
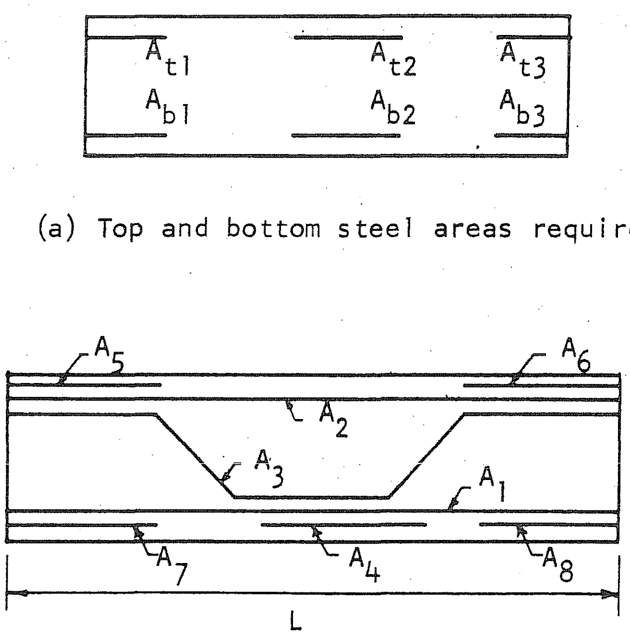


FIG. 3.15 TYPICAL SLAB



(a) Top and bottom steel areas required

(b) Layout of bars

Length of bars:

A_4 = 0.75L in column strips
= 0.70L in middle strips

$\left.\begin{matrix} A_5 \\ A_6 \\ A_7 \\ A_8 \end{matrix}\right\}$ = 0.30L in column strips
= 0.25L in middle strips

FIG. 3.16 STEEL AREAS REQUIRED AND LAYOUT OF BARS

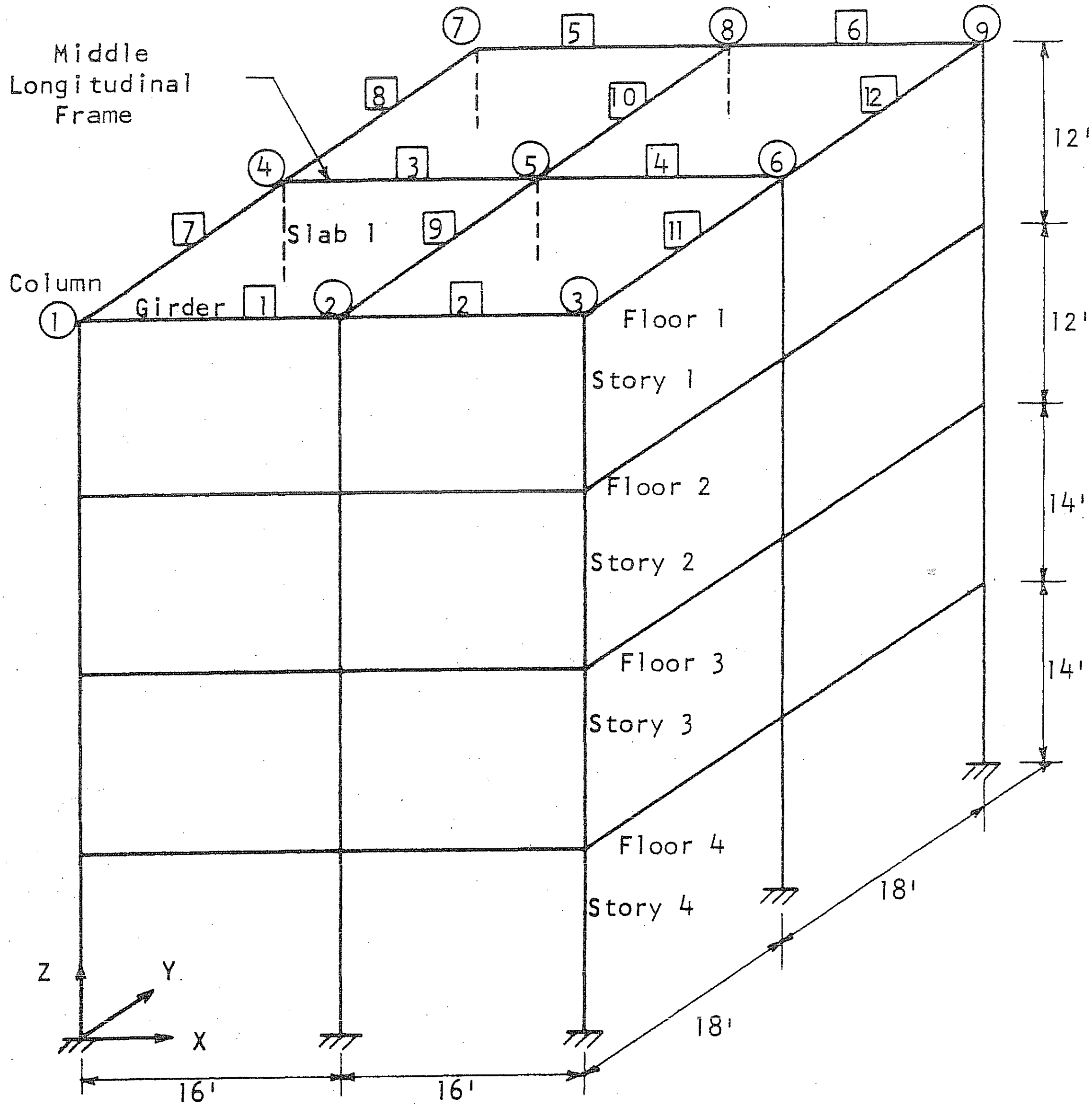
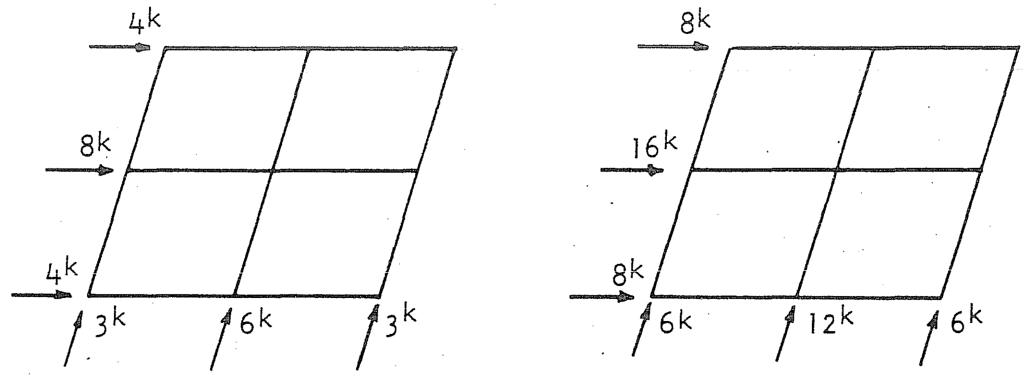
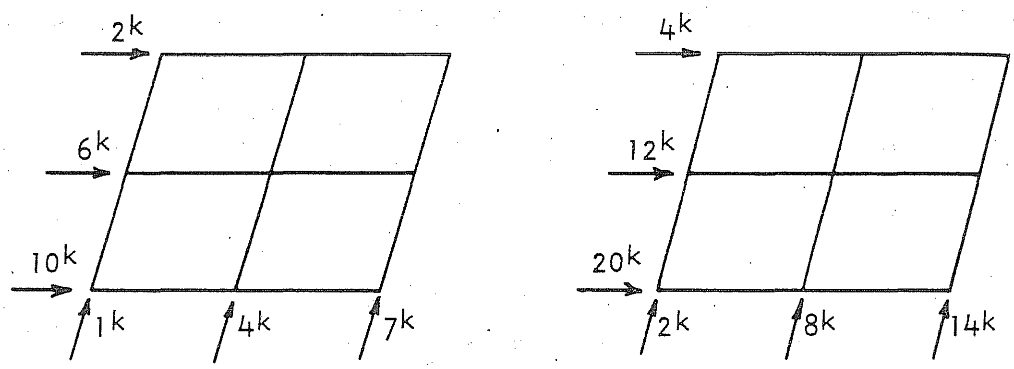


FIG. 4.1 EXAMPLE STRUCTURE



(a) Floor 1 (b) Floors 2,3,4

FIG. 4.2 SYMMETRICAL LATERAL LOAD



(a) Floor 1 (b) Floors 2,3,4

FIG. 4.3 UNSYMMETRICAL LATERAL LOAD

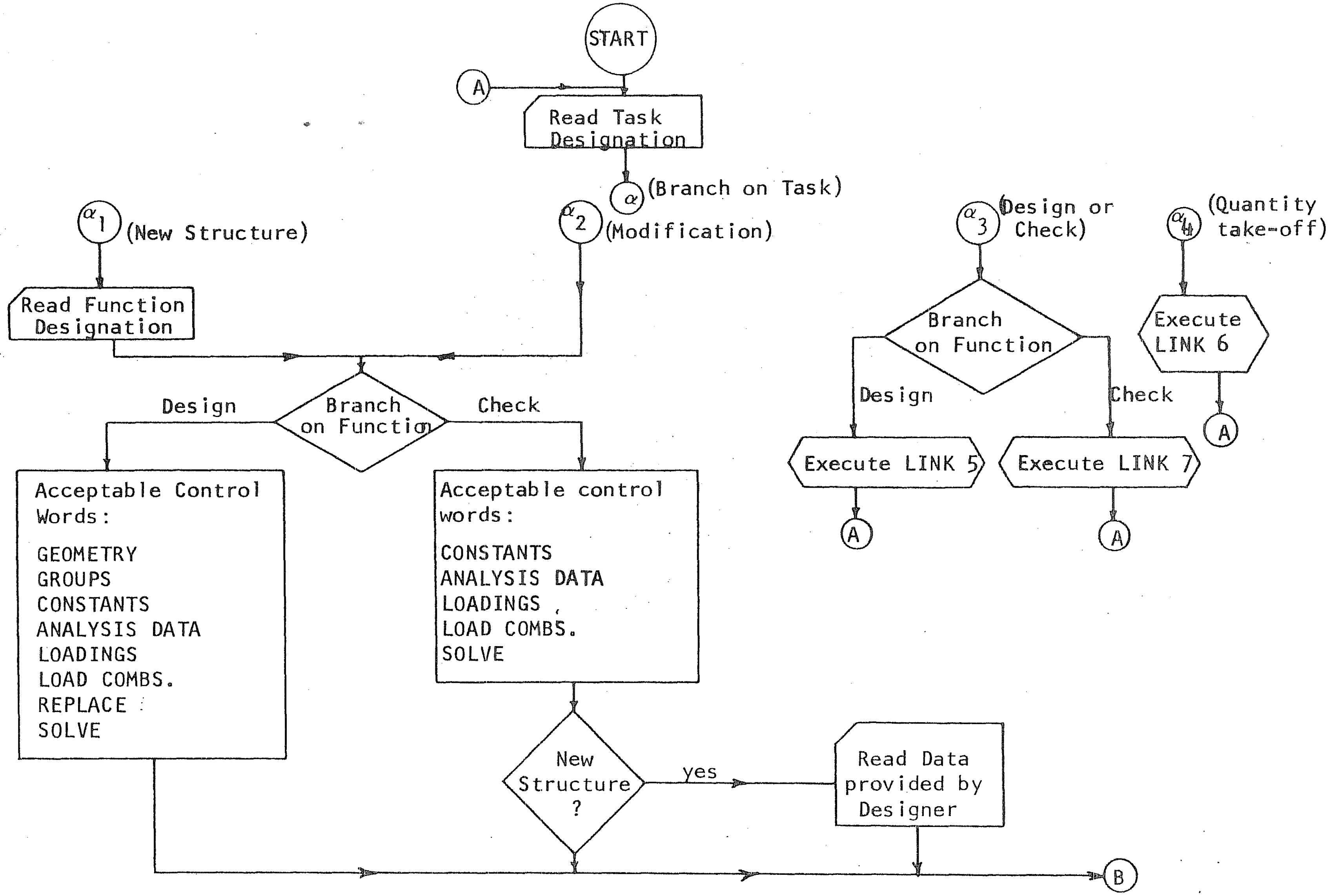


FIG. 5.1 BLOCK DIAGRAM OF COMPUTER PROGRAM

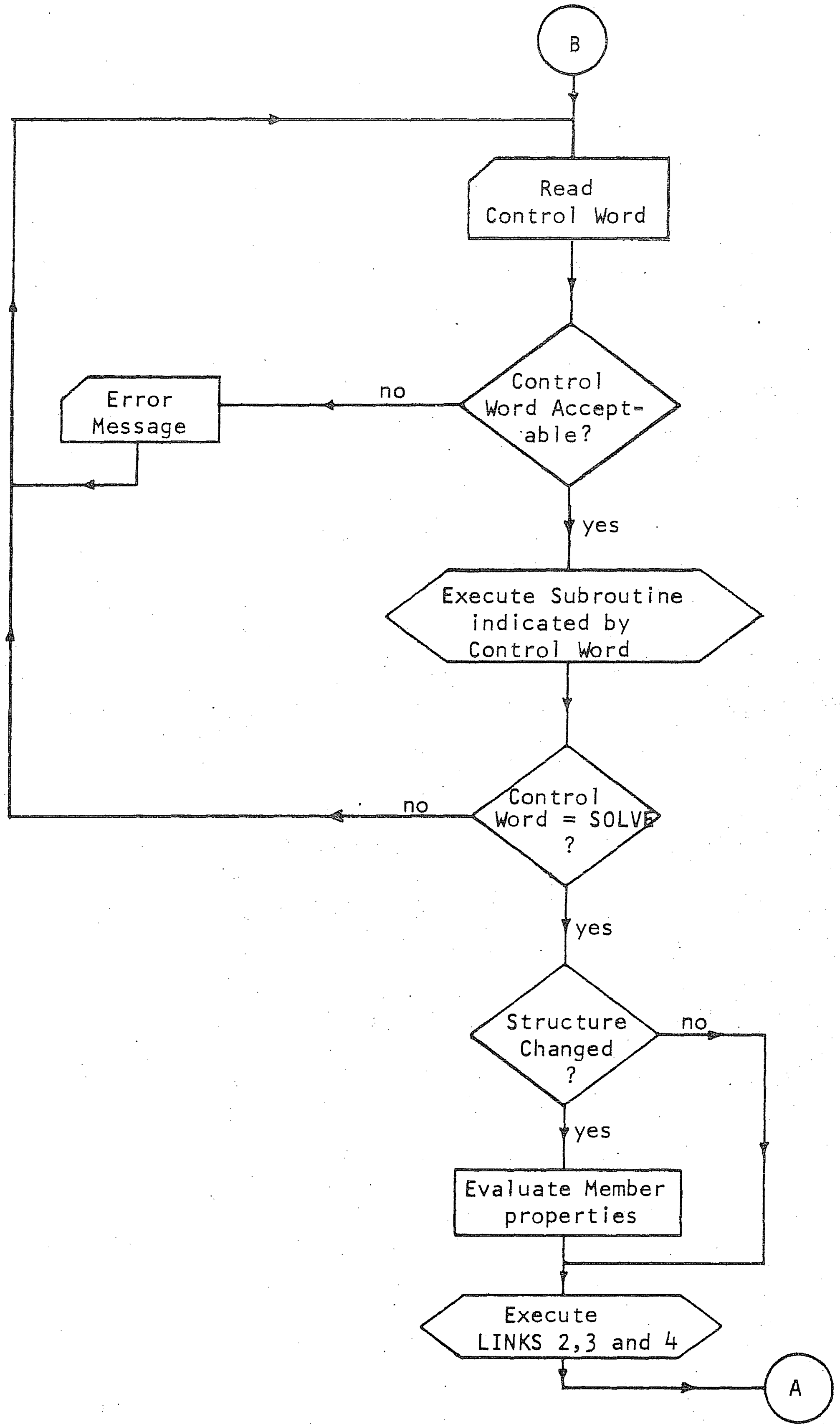
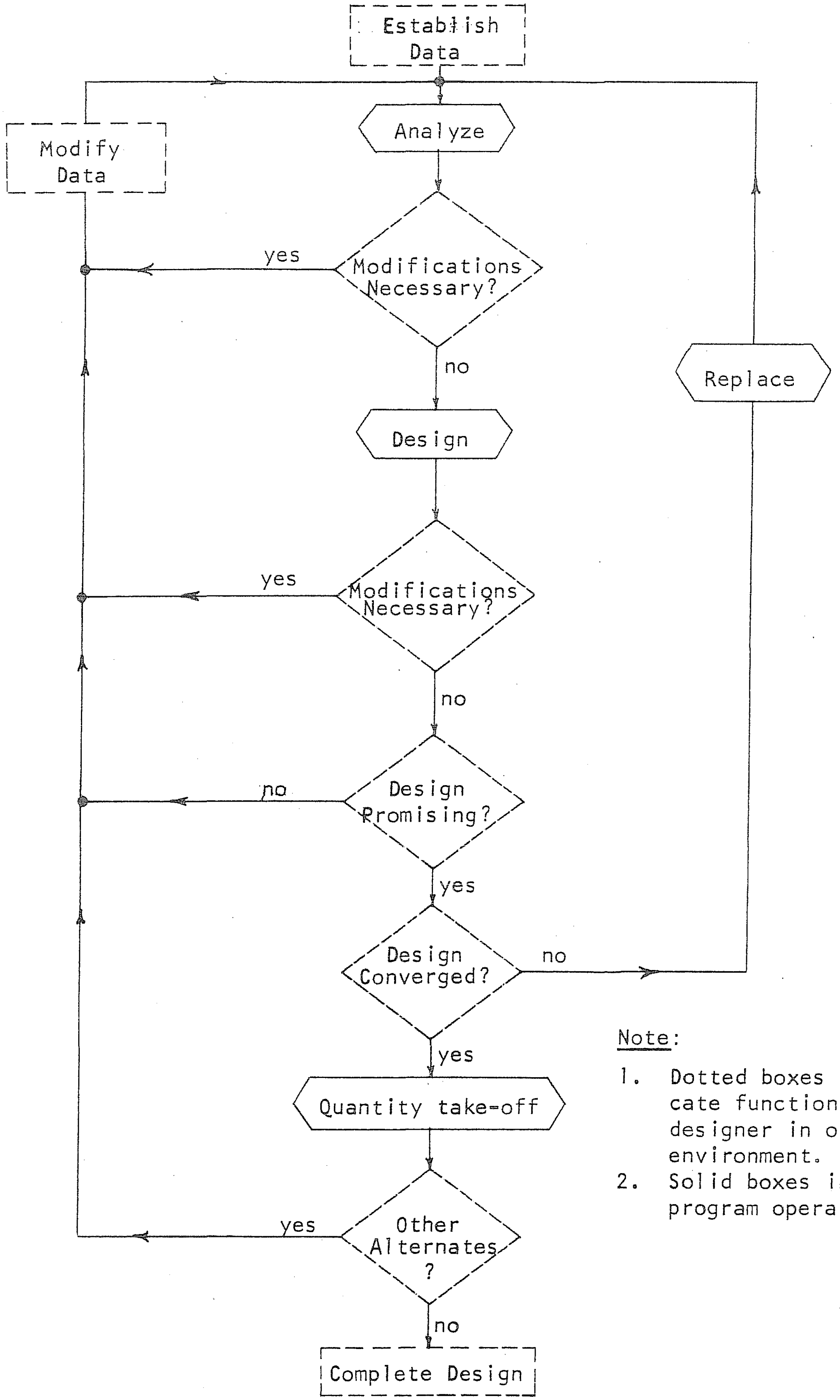


FIG. 5.1 (CONTINUED)



Note:

1. Dotted boxes indicate functions of designer in on-line environment.
2. Solid boxes indicate program operations.

FIG. 5.2 BLOCK DIAGRAM OF ANALYSIS AND DESIGN PROCESS

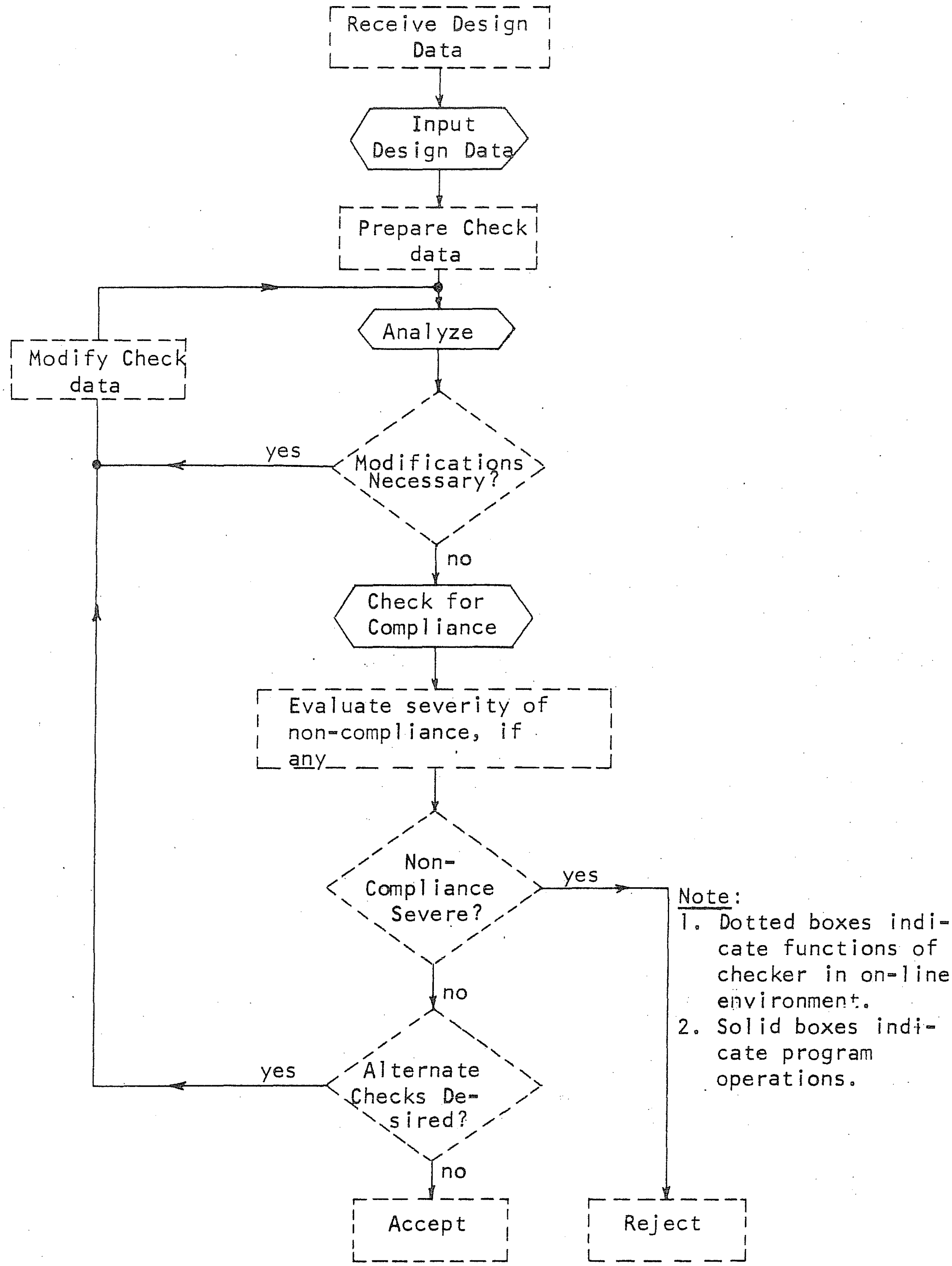


FIG. 5.3 BLOCK DIAGRAM OF CHECKING PROCESS

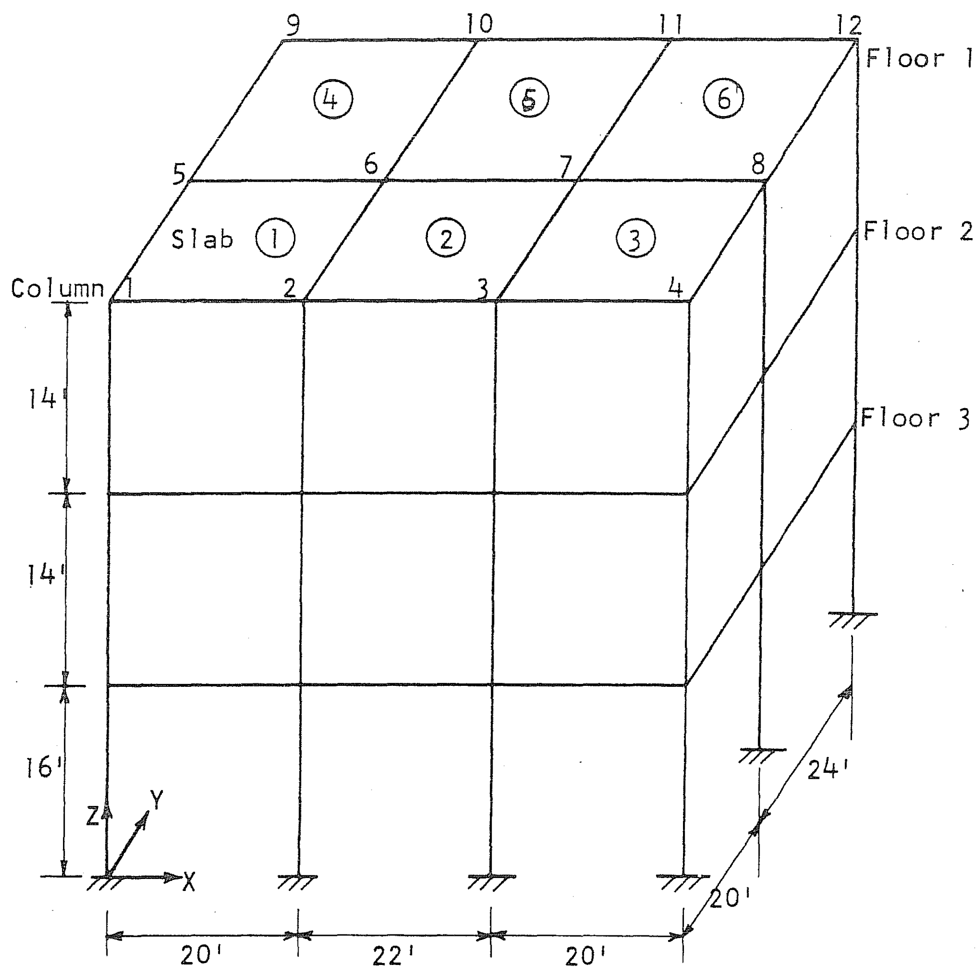


FIG. 6.1 EXAMPLE THREE STORY BUILDING

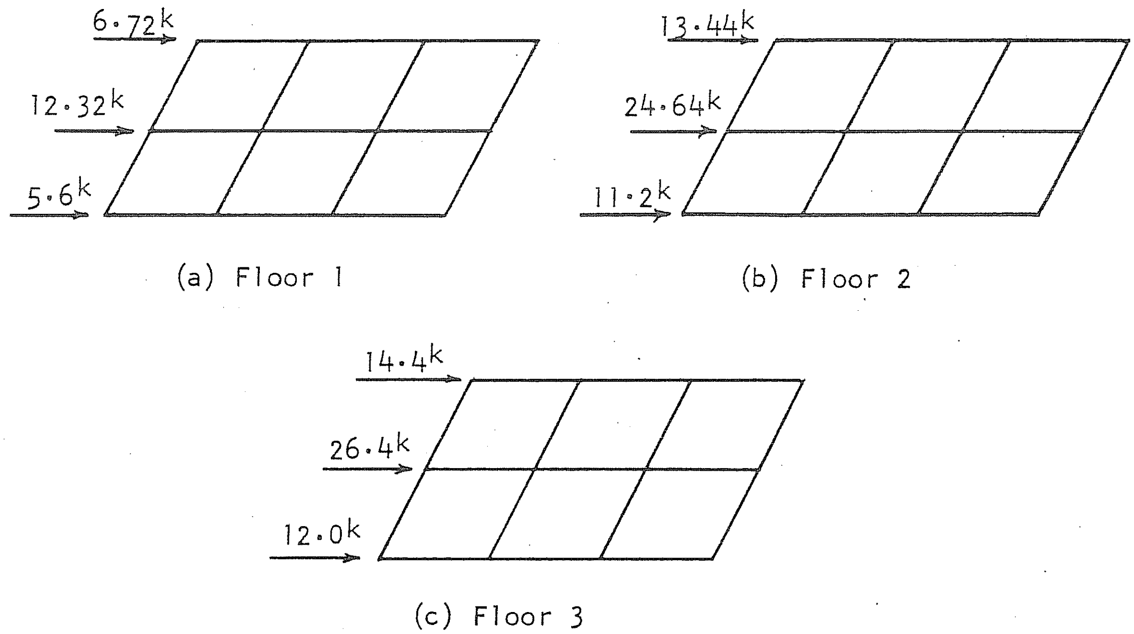


FIG. 6.2 LATERAL LOAD IN X DIRECTION

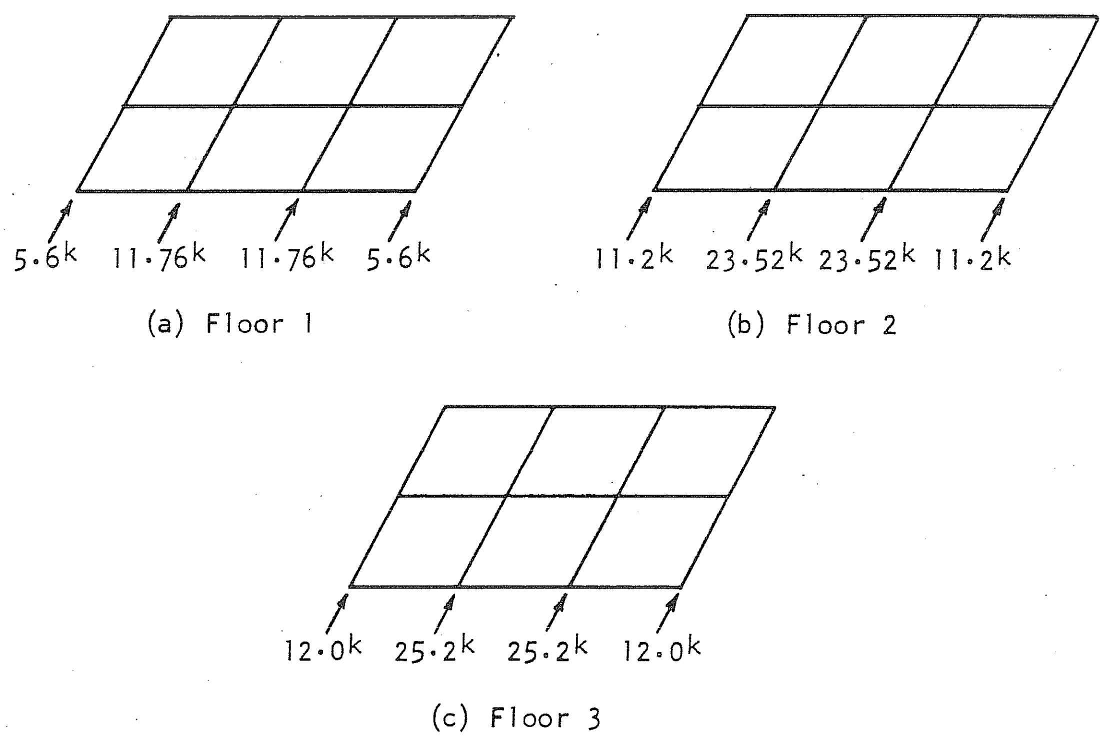
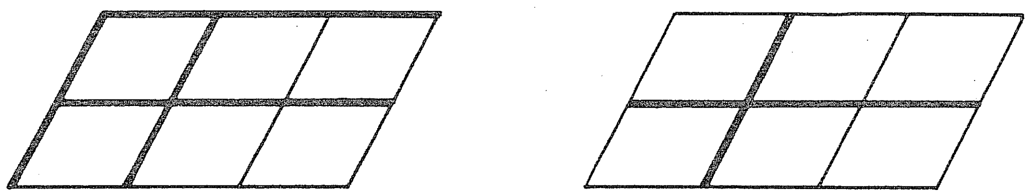


FIG. 6.3 LATERAL LOAD IN Y DIRECTION



(a) Design No. 7

(b) Design No. 8

FIG. 6.4 FRAMES ANALYZED FOR DESIGN NOS. 7 AND 8

Longitudinal Girders	Source List		1	7	6	4	3	8	3	2	6	5
	Sign and Multiplier List	K_{AA}	1	12	6	12	-6	1	-6	4	6	4
		K_{AB}	-1	-12	6	-12	-6	-1	6	2	-6	2
		K_{BA}	-1	-12	-6	-12	6	-1	-6	2	6	2
		K_{BB}	1	12	-6	12	6	1	6	4	-6	4
	Location List	Row	1	2	2	3	3	4	5	5	6	6
		Col.	1	2	6	3	5	4	3	5	2	6
Lateral Girders	Source List		7	6	1	4	3	3	2	8	6	5
	Sign and Multiplier List	K_{AA}	12	-6	1	12	6	6	4	1	-6	4
		K_{AB}	-12	-6	-1	-12	6	-6	2	-1	6	2
		K_{BA}	-12	6	-1	-12	-6	6	2	-1	-6	2
		K_{BB}	12	6	1	12	-6	-6	4	1	6	4
	Location List	Row	1	1	2	3	3	4	4	5	6	6
		Col.	1	6	2	3	4	3	4	5	1	6
Columns	Source List		4	3	7	6	1	6	5	3	2	8
	Sign and Multiplier List	K_{AA}	12	6	12	-6	1	-6	4	6	4	1
		K_{AB}	-12	6	-12	-6	-1	6	2	-6	2	-1
		K_{BA}	-12	-6	-12	6	-1	-6	2	6	2	-1
		K_{BB}	12	-6	12	6	1	6	4	-6	4	1
	Location List	Row	1	1	2	2	3	4	4	5	5	6
		Col.	1	5	2	4	3	2	4	1	5	6

TABLE 1. SPACE FRAME MEMBER LISTS FOR MAPPING FROM MEMBER TO JOINT MATRICES

<table>
<tr><td rowspan="8">Girders</td><td colspan="2">Source List</td><td>1</td><td>4</td><td>3</td><td>3</td><td>2</td></tr>
<tr><td rowspan="4">Sign and Multiplier List</td><td>K_{AA}</td><td>1</td><td>12</td><td>-6</td><td>-6</td><td>4</td></tr>
<tr><td>K_{AB}</td><td>-1</td><td>-12</td><td>-6</td><td>6</td><td>2</td></tr>
<tr><td>K_{BA}</td><td>-1</td><td>-12</td><td>6</td><td>-6</td><td>2</td></tr>
<tr><td>K_{BB}</td><td>1</td><td>12</td><td>6</td><td>6</td><td>4</td></tr>
<tr><td rowspan="2">Location List</td><td>Row</td><td>1</td><td>2</td><td>2</td><td>3</td><td>3</td></tr>
<tr><td>Col.</td><td>1</td><td>2</td><td>3</td><td>2</td><td>3</td></tr>
<tr><td colspan="2">Source List</td><td>4</td><td>3</td><td>1</td><td>3</td><td>2</td></tr>
<tr><td rowspan="6">Columns</td><td rowspan="4">Sign and Multiplier List</td><td>K_{AA}</td><td>12</td><td>6</td><td>1</td><td>6</td><td>4</td></tr>
<tr><td>K_{AB}</td><td>-12</td><td>6</td><td>-1</td><td>-6</td><td>2</td></tr>
<tr><td>K_{BA}</td><td>-12</td><td>-6</td><td>-1</td><td>6</td><td>2</td></tr>
<tr><td>K_{BB}</td><td>12</td><td>-6</td><td>1</td><td>-6</td><td>4</td></tr>
<tr><td rowspan="2">Location List</td><td>Row</td><td>1</td><td>1</td><td>2</td><td>3</td><td>3</td></tr>
<tr><td>Col.</td><td>1</td><td>3</td><td>2</td><td>1</td><td>3</td></tr>
</table>

TABLE 2. PLANE FRAME MEMBER LISTS FOR MAPPING FROM MEMBER TO JOINT MATRICES

ALTERNATIVE 1	ALTERNATIVE 2
<u>Source Array</u> $\begin{bmatrix} 1,1 & 1,2 & 1,3 \\ 2,1 & 2,2 & 2,3 \\ 3,1 & 3,2 & 3,3 \end{bmatrix}$ <u>Destination Array</u> (n=3) $\begin{bmatrix} JJ+1,JK+1 & JJ+1,JK+2 & JJ+1,JK+3 \\ JJ+2,JK+1 & JJ+2,JK+2 & JJ+2,JK+3 \\ JJ+3,JK+1 & JJ+3,JK+2 & JJ+3,JK+3 \end{bmatrix}$	<u>Source Array</u> $\begin{bmatrix} 1,1 & 1,3 \\ 3,1 & 3,3 \end{bmatrix}$ <u>Destination Array</u> (n=2) $\begin{bmatrix} JJ+1,JK+1 & JJ+1,JK+2 \\ JJ+2,JK+1 & JJ+2,JK+2 \end{bmatrix}$
ALTERNATIVE 3	**ALTERNATIVE 4**
<u>Source Array</u> Same as for Alternative 1 <u>Destination Array</u> (n=2) $\begin{bmatrix} NK+1,NK+1 & NK+1,JK+1 & NK+1,JK+2 \\ JJ+1,NK+1 & JJ+1,JK+1 & JJ+1,JK+2 \\ JJ+2,NK+1 & JJ+2,JK+1 & JJ+2,JK+2 \end{bmatrix}$	<u>Source Array</u> Same as for Alternative 2 <u>Destination Array</u> (n=1) $\begin{bmatrix} NK+1,NK+1 & NK+1,JK+1 \\ JJ+1,NK+1 & JJ+1,JK+1 \end{bmatrix}$
ALTERNATIVE 5	**ALTERNATIVE 6**
<u>Source Array</u> [3,3] <u>Destination Array</u> (n=1) [JJ+1,JK+1]	<u>Source Array</u> [1,1] <u>Destination Array</u> [1,1]
DEFINITION OF SYMBOLS	
J = Joint number NJ = Number of joints per story n = Number of independent deformation components per joint NK = NJ x n Diagonal Submatrices: JJ=JK=(J-1) x n Off-diagonal Submatrices: JJ=(J-1) x n; JK=J x n	

TABLE 3. MAPPING FROM JOINT TO STRUCTURE MATRIX FOR PLANE FRAME ALTERNATIVES

ALTERNATIVE 1

Source Array

$$\begin{bmatrix} 1,1 & 1,2 & 1,3 & 1,4 & 1,5 & 1,6 \\ 2,1 & 2,2 & 2,3 & 2,4 & 2,5 & 2,6 \\ 3,1 & 3,2 & 3,3 & 3,4 & 3,5 & 3,6 \\ 4,1 & 4,2 & 4,3 & 4,4 & 4,5 & 4,6 \\ 5,1 & 5,2 & 5,3 & 5,4 & 5,5 & 5,6 \\ 6,1 & 6,2 & 6,3 & 6,4 & 6,5 & 6,6 \end{bmatrix}$$

Destination Array (n=6)

$$\begin{bmatrix} JJ+1,JK+1 & JJ+1,JK+2 & JJ+1,JK+3 & JJ+1,JK+4 & JJ+1,JK+5 & JJ+1,JK+6 \\ JJ+2,JK+1 & JJ+2,JK+2 & JJ+2,JK+3 & JJ+2,JK+4 & JJ+2,JK+5 & JJ+2,JK+6 \\ JJ+3,JK+1 & JJ+3,JK+2 & JJ+3,JK+3 & JJ+3,JK+4 & JJ+3,JK+5 & JJ+3,JK+6 \\ JJ+4,JK+1 & JJ+4,JK+2 & JJ+4,JK+3 & JJ+4,JK+4 & JJ+4,JK+5 & JJ+4,JK+6 \\ JJ+5,JK+1 & JJ+5,JK+2 & JJ+5,JK+3 & JJ+5,JK+4 & JJ+5,JK+5 & JJ+5,JK+6 \\ JJ+6,JK+1 & JJ+6,JK+2 & JJ+6,JK+3 & JJ+6,JK+4 & JJ+6,JK+5 & JJ+6,JK+6 \end{bmatrix}$$

ALTERNATIVE 2

Source Array

$$\begin{bmatrix} 1,1 & 1,2 & 1,4 & 1,5 & 1,6 \\ 2,1 & 2,2 & 2,4 & 2,5 & 2,6 \\ 4,1 & 4,2 & 4,4 & 4,5 & 4,6 \\ 5,1 & 5,2 & 5,4 & 5,5 & 5,6 \\ 6,1 & 6,2 & 6,4 & 6,5 & 6,6 \end{bmatrix}$$

Destination Array (n=5)

$$\begin{bmatrix} JJ+1,JK+1 & JJ+1,JK+2 & JJ+1,JK+3 & JJ+1,JK+4 & JJ+1,JK+5 \\ JJ+2,JK+1 & JJ+2,JK+2 & JJ+2,JK+3 & JJ+2,JK+4 & JJ+2,JK+5 \\ JJ+3,JK+1 & JJ+3,JK+2 & JJ+3,JK+3 & JJ+3,JK+4 & JJ+3,JK+5 \\ JJ+4,JK+1 & JJ+4,JK+2 & JJ+4,JK+3 & JJ+4,JK+4 & JJ+4,JK+5 \\ JJ+5,JK+1 & JJ+5,JK+2 & JJ+5,JK+3 & JJ+5,JK+4 & JJ+5,JK+5 \end{bmatrix}$$

ALTERNATIVE 3

Source Array

$$\begin{bmatrix} 1,1 & 1,2 & 1,3 & 1,4 & 1,5 \\ 2,1 & 2,2 & 2,3 & 2,4 & 2,5 \\ 3,1 & 3,2 & 3,3 & 3,4 & 3,5 \\ 4,1 & 4,2 & 4,3 & 4,4 & 4,5 \\ 5,1 & 5,2 & 5,3 & 5,4 & 5,5 \end{bmatrix}$$

Destination Array (n=5)

Same as for Space frame Alternative 2

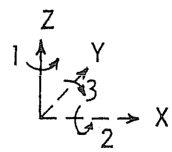
TABLE 4. MAPPING FROM JOINT TO STRUCTURE MATRIX FOR SPACE FRAME ALTERNATIVES

ALTERNATIVE 4

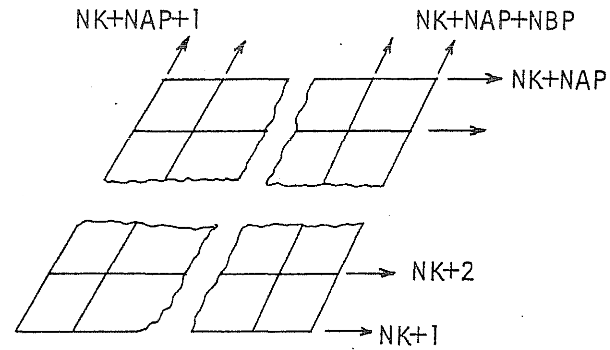
Source Array

Same as for space frame Alternative 1.

Destination Array (n=4)



(a) Independent Joint Deformations



(b) Additional Floor Deformations

With NX = NK+LFNO and NY = NK+NAP+LATFNO, the destination array is:

$$\begin{bmatrix}
NX,NX & NX,NY & NX,JK+1 & NX,JK+2 & NX,JK+3 & NX,JK+4 \\
NY,NX & NY,NY & NY,JK+1 & NY,JK+2 & NY,JK+3 & NY,JK+4 \\
JJ+1,NX & JJ+1,NY & JJ+1,JK+1 & JJ+1,JK+2 & JJ+1,JK+3 & JJ+1,JK+4 \\
JJ+2,NX & JJ+2,NY & JJ+2,JK+1 & JJ+2,JK+2 & JJ+2,JK+3 & JJ+2,JK+4 \\
JJ+3,NX & JJ+3,NY & JJ+3,JK+1 & JJ+3,JK+2 & JJ+3,JK+3 & JJ+3,JK+4 \\
JJ+4,NX & JJ+4,NY & JJ+4,JK+1 & JJ+4,JK+2 & JJ+4,JK+3 & JJ+4,JK+4
\end{bmatrix}$$

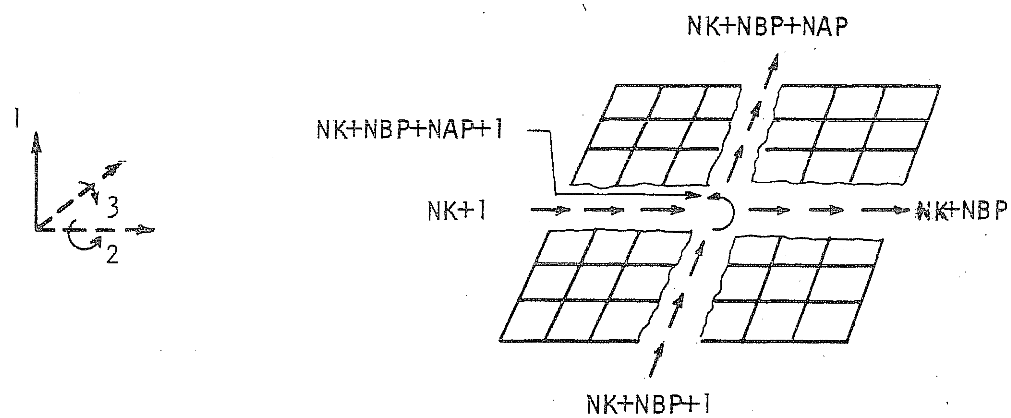
TABLE 4. (Continued)

ALTERNATIVE 5

Source Array

Same as for space frame Alternative 1

Destination Array (n=3)



(a) Independent Joint Deformations

(b) Additional Floor Deformations

With NX = NK+LATFNO, NY = NK+NBP+LFNO and NZ = NK+NBP+NAP+1, the destination array is:

$$\begin{bmatrix} NX,NX & NX,NY & NX,JK+1 & NX,JK+2 & NX,JK+3 & NX,NZ \\ NY,NX & NY,NY & NY,JK+1 & NY,JK+2 & NY,JK+3 & NY,NZ \\ JJ+1,NX & JJ+1,NY & JJ+1,JK,1 & JJ+1,JK+2 & JJ+1,JK+3 & JJ+1,NZ \\ JJ+2,NX & JJ+2,NY & JJ+2,JK+1 & JJ+2,JK+2 & JJ+2,JK+3 & JJ+2,NZ \\ JJ+3,NX & JJ+3,NY & JJ+3,JK+1 & JJ+3,JK+2 & JJ+3,JK+3 & JJ+3,NZ \\ NZ,NX & NZ,NY & NZ,JK+1 & NZ,JK+2 & NZ,JK+3 & NZ,NZ \end{bmatrix}$$

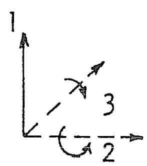
TABLE 4. (Continued)

ALTERNATIVE 6

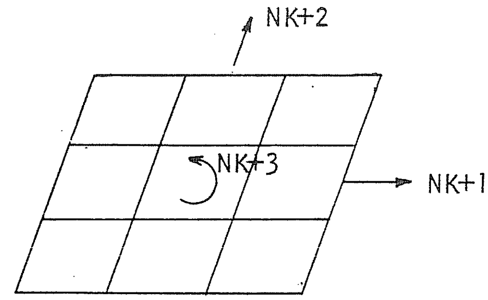
Source Array

Same as for space frame Alternative 1.

Destination Array (n=3)



(a) Independent Joint Deformations



(b) Additional Floor Deformations

The destination array is the same as for space frame Alternative 5 with NX = NK+1, NY = NK+2 and NZ = NK+3.

ALTERNATIVE 7

This is a combination of space frame Alternatives 2 & 4.

Source Array

Same as for space frame Alternative 2.

Destination Array (n=3)

With NX = NK+LFNO and NY = NK+NAP+LATFNO, the destination array is:

$$\begin{bmatrix} NX,NX & NX,NY & NX,JK+1 & NX,JK+2 & NX,JK+3 \\ NY,NX & NY,NY & NY,JK+1 & NY,JK+2 & NY,JK+3 \\ JJ+1,NX & JJ+1,NY & JJ+1,JK+1 & JJ+1,JK+2 & JJ+1,JK+3 \\ JJ+2,NX & JJ+2,NY & JJ+2,JK+1 & JJ+2,JK+2 & JJ+2,JK+3 \\ JJ+3,NX & JJ+3,NY & JJ+3,JK+1 & JJ+3,JK+2 & JJ+3,JK+3 \end{bmatrix}$$

TABLE 4. (Continued)

ALTERNATIVE 8

This is a combination of space frame Alternatives 2 & 5.

Source Array

Same as for space frame Alternative 2.

Destination Array (n=2)

With NX = NK+LATFNO, NY = NK+NBP+LFNO and NZ = NK+NBP+NAP+1, the destination array is:

$$\begin{bmatrix} NX,NX & NX,NY & NX,JK+1 & NX,JK+2 & NX,NZ \\ NY,NX & NY,NY & NY,JK+1 & NY,JK+2 & NY,NZ \\ JJ+1,NX & JJ+1,NY & JJ+1,JK+1 & JJ+1,JK+2 & JJ+1,NZ \\ JJ+2,NX & JJ+2,NY & JJ+2,JK+1 & JJ+2,JK+2 & JJ+2,NZ \\ NZ,NX & NZ,NY & NZ,JK+1 & NZ,JK+2 & NZ,NZ \end{bmatrix}$$

ALTERNATIVE 9

This is a combination of space frame Alternatives 2 & 6.

Source Array

Same as for space frame Alternative 2.

Destination Array (n=2)

The destination array is the same as for space frame Alternative 8 with NX = NK+1, NY = NK+2 and NZ = NK+3.

TABLE 4. (Continued)

ALTERNATIVE 10
Source Array $$\begin{bmatrix} 4,4 & 4,5 \\ 5,4 & 5,5 \end{bmatrix}$$ Destination Array (n=2) $$\begin{bmatrix} JJ+1,JK+1 & JJ+1,JK+2 \\ JJ+2,JK+1 & JJ+2,JK+2 \end{bmatrix}$$

DEFINITION OF SYMBOLS
J = Joint number NJ = Number of joints per story. n = Number of independent deformation components per joint. NK = NJ x n LFNO,LATFNO = Longitudinal and lateral frame numbers intersecting at joint J. NAP = Number of longitudinal frames. NBP = Number of lateral frames Diagonal Submatrices: JJ = JK = (J-1) x n Off-diagonal Submatrices: JJ = (J-1) x n JK = J x n for first off-diagonal submatrix JK = (J+NBAY) x n for second off-diagonal submatrix. NBAY = Number of bays.

TABLE 4. (Continued)

Alternative		Source (S) and Destination (D) Lists					
2	S	JJ+1	JJ+2	JJ+4	JJ+5	JJ+6	
	D (n=5)	JK+1	JK+2	JK+3	JK+4	JK+5	
3	S	JJ+1	JJ+2	JJ+3	JJ+4	JJ+5	
	D (n=5)	JK+1	JK+2	JK+3	JK+4	JK+5	
4	S	JJ+1	JJ+2	JJ+3	JJ+4	JJ+5	JJ+6
	D (n=4)	NK+LFNO	NK+NAP +LATFNO	JK+1	JK+2	JK+3	JK+4
5	S	JJ+1	JJ+2	JJ+3	JJ+4	JJ+5	JJ+6
	D (n=3)	NK+LATFNO	NK+NBP +LFNO	JK+1	JK+2	JK+3	NK+NBP +NAP+1
6	S	JJ+1	JJ+2	JJ+3	JJ+4	JJ+5	JJ+6
	D (n=3)	NK+1	NK+2	JK+1	JK+2	JK+3	NK+3
7	S	JJ+1	JJ+2	JJ+4	JJ+5	JJ+6	
	D (n=3)	NK+LFNO	NK+NAP +LATFNO	JK+1	JK+2	JK+3	
8	S	JJ+1	JJ+2	JJ+4	JJ+5	JJ+6	
	D (n=2)	NK+ LATFNO	NK+NBP +LFNO	JK+1	JK+2	NK+NBP +NAP+1	
9	S	JJ+1	JJ+2	JJ+4	JJ+5	JJ+6	
	D (n=2)	NK+1	NK+2	JK+1	JK+2	NK+3	
10	S	JJ+4	JJ+5				
	D (n=2)	JK+1	JK+2				

DEFINITION OF SYMBOLS

J = Joint number

NJ = Number of joints per story

n = Number of independent deformation components per joint

NK = NJ x n

LFNO, LATFNO = Longitudinal and lateral frame numbers intersecting at joint J

NAP = Number of longitudinal frames

NBP = Number of lateral frames

JJ = (J-1) x 6

JK = (J-1) x n

TABLE 5. LISTS FOR MAPPING JOINT LOADS FROM SPACE FRAME ALTERNATIVE 1 TO OTHER SPACE FRAME ALTERNATIVES

Alternative	Source (S) and Destination (D) Lists		
		Longitudinal Frames	Lateral Frames
1	S D (n=3)	JJ+1 JJ+3 JJ+5 JK+1 JK+2 JK+3	JJ+2 JJ+3 JJ+4 JK+1 JK+2 JK+3
2	S D (n=2)	JJ+1 JJ+5 JK+1 JK+2	JJ+2 JJ+4 JK+1 JK+2
3	S D (n=2)	JJ+1 JJ+3 JJ+5 NK+1 JK+1 JK+2	JJ+2 JJ+3 JJ+4 NK+1 JK+1 JK+2
4	S D (n=1)	JJ+1 JJ+5 NK+1 JK+1	JJ+2 JJ+4 NK+1 JK+1
5	S D (n=1)	JJ+5 JK+1	JJ+4 JK+1
6	S D (n=0)	1 1	1 1

DEFINITION OF SYMBOLS

J = Joint number

n = Number of independent deformation components per joint

NJ = Number of joints per story

NK = NJ x n

JJ = (J-1) x 6

JK = (J-1) x n

TABLE 6. LISTS FOR MAPPING JOINT LOADS FROM SPACE FRAME ALTERNATIVE 1 TO PLANE FRAME ALTERNATIVES

Alternative	Source (S) and Destination (D) Lists						
1	S (n=6)	JJ+1	JJ+2	JJ+3	JJ+4	JJ+5	JJ+6
	D	KK+1	KK+2	KK+3	KK+4	KK+5	KK+6
2	S (n=5)	JJ+1	JJ+2	JJ+3	JJ+4	JJ+5	
	D	KK+1	KK+2	KK+4	KK+5	KK+6	
3	S (n=5)	JJ+1	JJ+2	JJ+3	JJ+4	JJ+5	
	D	KK+1	KK+2	KK+3	KK+4	KK+5	
4	S (n=4)	NK+LFNO	NK+NAP+ LATFNO	JJ+1	JJ+2	JJ+3	JJ+4
	D	KK+1	KK+2	KK+3	KK+4	KK+5	KK+6
5	S (n=3)	NK+LATFNO	NK+NBP+ LFNO	JJ+1	JJ+2	JJ+3	NK+NBP+ NAP+1
	D	KK+1	KK+2	KK+3	KK+4	KK+5	KK+6
6	S (n=3)	NK+1	NK+2	JJ+1	JJ+2	JJ+3	NK+3
	D	KK+1	KK+2	KK+3	KK+4	KK+5	KK+6
7	S (n=3)	NK+LFNO	NK+NAP+ LATFNO	JJ+1	JJ+2	JJ+3	
	D	KK+1	KK+2	KK+4	KK+5	KK+6	
8	S (n=2)	NK+LATFNO	NK+NBP +LFNO	JJ+1	JJ+2	NK+NBP+ NAP+1	
	D	KK+1	KK+2	KK+4	KK+5	KK+6	
9	S (n=2)	NK+1	NK+2	JJ+1	JJ+2	NK+3	
	D	KK+1	KK+2	KK+4	KK+5	KK+6	
10	S (n=2)	JJ+1	JJ+2				
	D	KK+4	KK+5				

DEFINITION OF SYMBOLS

J = Joint number

NJ = Number of joints per story

n = Number of independent deformation components per joint

NK = NJ x n

LFNO, LATFNO = Longitudinal and lateral frame numbers intersecting at joint J

TABLE 7. MAPPING FROM JOINT DEFORMATIONS TO MEMBER DEFORMATIONS FOR SPACE FRAME ALTERNATIVES

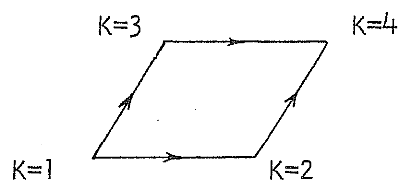
DEFINITION OF SYMBOLS (Cont.)

NAP = Number of longitudinal frames

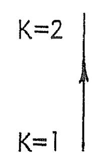
NBP = Number of lateral frames

JJ = (J-1) x n

KK = (K-1) x 6, where K is as shown in Figs. (a) and (b) for slabs and columns respectively.



(a) 'K' for Slabs



(b) 'K' for Columns

TABLE 7. (Continued)

Alternative		Source (S) and Destination (D) Lists		
1	S (n=3)	JJ+1	JJ+2	JJ+3
	D	KK+1	KK+2	KK+3
2	S (n=2)	JJ+1	JJ+2	
	D	KK+1	KK+3	
3	S (n=2)	NK+1	JJ+1	JJ+2
	D	KK+1	KK+2	KK+3
4	S (n=1)	NK+1	JJ+1	
	D	KK+1	KK+3	
5	S (n=1)	JJ+1		
	D	KK+3		
6	S (n=0)	1		
	D	1		

DEFINITION OF SYMBOLS

J = Joint number

n = Number of independent deformation components per joint

NJ = Number of joints per story

NK = NK x n

JJ = (J-1) x n

KK = (K-1) x 3, where K = 1 for positive node of member, and K = 2 for negative node of member

TABLE 8. MAPPING FROM JOINT DEFORMATIONS TO MEMBER DEFORMATIONS FOR PLANE FRAME ALTERNATIVES

Loading Unit	Rule for Accumulating Combined Stress Resultants
1 Non-reversible loading, a	Rule: The stress resultant is added to the corresponding* CSR+ of the same sign as the stress resultant. Exception: For dead loads, the stress resultant is added to the corresponding CSR's of both signs.
2 Reversible loading, ±a	Rule: The numerical value of the stress resultant is added to the corresponding CSR's of both signs with the sign of the CSR.
3 a OR b	Rule: If the stress resultants due to loadings a and b are of the same sign, the numerically larger stress resultant is added to the corresponding CSR of the same sign. If the stress resultants are of opposite signs, the positive stress resultant is added to the corresponding positive CSR and the negative stress resultant is added to the corresponding negative CSR.
4 a OR ± b	Rule: The absolute values of the stress resultants due to loadings a and b are compared. If the stress resultant due to loading a is numerically larger, it is added to the corresponding CSR of the same sign and the absolute value of the stress resultant due to loading b is added to the corresponding CSR of the opposite sign, with the sign of the CSR. If the stress resultant due to loading b is numerically equal or larger, the absolute value of the stress resultant is added to the corresponding CSR's of both signs, with the sign of the CSR.
5 ±a OR ±b	Rule: The absolute values of the stress resultants due to loadings a and b are compared, and the numerically larger value is added to the corresponding CSR's of both signs, with the sign of the CSR.

* Corresponding indicates same location

\+ CSR stands for Combined Stress Resultant

TABLE 9. LOADING COMBINATIONS FOR GIRDERS

Criterion for set of quantities	LOADING UNIT					
	1 Non-reversible loading, a		3 Non-reversible loading a OR Reversible loading $\pm$ b			
Maximum P	Check P_a		Check P_a,P_b			
	P_a positive	P_a negative	Both positive		One positive	
			P_a larger	P_b larger	P_a positive	P_b positive
	Add a	No Contribution	Add a	Add b	Add a	Add b
Maximum Positive M_x	Check M_{xa}		Check M_{xa},M_{xb}			
	M_{xa} positive	M_{xa} negative	Both positive		One positive	
			M_{xa} larger	M_{xb} larger	M_{xa} positive	M_{xb} positive
	Add a	No Contribution	Add a	Add b	Add a	Add b

TABLE 10. LOADING COMBINATIONS FOR COLUMNS (LOADING UNITS 1,3,4 and 5)

Criterion for set of quantities	LOADING UNIT						
	4 a OR $\pm$ b			5 $\pm$ a OR $\pm$ b			
Maximum P	Compare $\lvert P_a \rvert$ and $\lvert P_b \rvert$			Compare $\lvert P_a \rvert$ and $\lvert P_b \rvert$			
	$\lvert P_a \rvert$ larger and P_a positive	$\lvert P_b \rvert$ larger or P_a negative		$\lvert P_a \rvert$ larger		$\lvert P_b \rvert$ larger	
		P_b positive	P_b negative	P_a positive	P_a negative	P_b positive	P_b negative
	Add a	Add b	Subtract b	Add a	Subtract a	Add b	Subtract b
Maximum Positive M_x	Compare $\lvert M_{xa} \rvert$ and $\lvert M_{xb} \rvert$			Compare $\lvert M_{xa} \rvert$ and $\lvert M_{xb} \rvert$			
	$\lvert M_{xa} \rvert$ larger and M_{xa} positive	$\lvert M_{xb} \rvert$ larger or M_{xa} negative		$\lvert M_{xa} \rvert$ larger		$\lvert M_{xb} \rvert$ larger	
		M_{xb} positive	M_{xb} negative	M_{xa} positive	M_{xa} negative	M_{xb} positive	M_{xb} negative
	Add a	Add b	Subtract b	Add a	Subtract a	Add b	Subtract b

TABLE 10. (Continued)

<table>
<tr><td rowspan="2">Criterion for set of quantities</td><td colspan="8">Loading Unit</td></tr>
<tr><td colspan="8">2
Reversible Loading, ±a</td></tr>
<tr><td rowspan="7">Maximum P</td><td colspan="8">Check P_a</td></tr>
<tr><td colspan="2">P_a positive</td><td colspan="2">P_a negative</td><td colspan="4">$P_a = 0$</td></tr>
<tr><td colspan="2" rowspan="5">Add a</td><td colspan="2" rowspan="5">Subtract a</td><td colspan="2">Add a</td><td colspan="2">Subtract a</td></tr>
<tr><td colspan="2">$SUM_a = |M_x| + |M_y|$</td><td colspan="2">$SUM_b = |M_x| + |M_y|$</td></tr>
<tr><td colspan="4">Compare SUM_a and SUM_b</td></tr>
<tr><td colspan="2">SUM_a larger</td><td colspan="2">SUM_b larger</td></tr>
<tr><td colspan="2">Add a</td><td colspan="2">Subtract a</td></tr>
<tr><td rowspan="7">Maximum Positive P</td><td colspan="8">Check M_{xa}</td></tr>
<tr><td colspan="4">$M_{xa} \neq 0$</td><td colspan="4">$M_{xa} = 0$</td></tr>
<tr><td colspan="2">Add a</td><td colspan="2">Subtract a</td><td colspan="4">Check M_y</td></tr>
<tr><td colspan="2">$SUM_a = |M_x| + |M_y|$</td><td colspan="2">$SUM_b = |M_x| + |M_y|$</td><td colspan="2">M_y positive</td><td colspan="2">M_y negative</td></tr>
<tr><td colspan="4">Compare SUM_a and SUM_b</td><td colspan="4">Check M_{ya}</td></tr>
<tr><td colspan="2">SUM_a larger</td><td colspan="2">SUM_b larger</td><td>M_{ya} pos.</td><td>M_{ya} neg.</td><td>M_{ya} pos.</td><td>M_{ya} neg.</td></tr>
<tr><td colspan="2">Add a</td><td colspan="2">Subtract a</td><td>Add a</td><td>Sub. a</td><td>Sub. a</td><td>Add a</td></tr>
</table>

TABLE 11. LOADING COMBINATIONS FOR COLUMNS (LOADING UNIT 2)

Analysis Alternative		LOADING												
		Dead Load			Full Live Load			Live Load Maxima			Sym. Lateral Ld.		Unsym. Lateral Ld.	
		M_A	M_C	M_B	M_A	M_C	M_B	M_A	M_C	M_B	M_A	M_B	M_A	M_B
Space Frame Alternatives								(1.3	-3.2	3.3)				
	1	-42.1	27.0	-47.5	-27.1	17.4	-30.4	-28.4	20.5	-33.7	24.2	-22.0	27.3	-24.8
	2	-37.1	26.8	-53.2	-23.8	17.2	-34.2	-26.2	20.4	-35.9	24.6	-22.3	27.7	-25.2
	3	-42.1	27.0	-47.5	-27.1	17.4	-30.4	-28.4	20.5	-33.7	24.2	-22.0	27.2	-24.8
	4	-42.2	27.0	-47.5	-27.1	17.3	-30.4	-28.4	20.5	-33.7	24.2	-22.0	27.3	-24.8
	5	-42.1	27.0	-47.5	-27.1	17.4	-30.4	-28.4	20.5	-33.7	24.2	-22.0	27.2	-24.8
	6	-42.2	27.0	-47.5	-27.1	17.3	-30.4	-28.4	20.5	-33.7	24.2	-22.0	27.2	-24.8
	7	-37.1	26.8	-53.2	-23.8	17.2	-34.2	-26.3	20.4	-35.9	24.6	-22.4	27.7	-25.2
	8	-37.1	26.8	-53.2	-23.8	17.2	-34.2	-26.2	20.4	-35.9	24.6	-22.4	27.7	-25.2
								(2.5	-3.3	+1.7)				
	9	-37.1	26.8	-53.2	-23.8	17.2	-34.2	-26.3	20.4	-35.9	24.6	-22.4	27.7	-25.2
	10	-37.1	26.8	-53.2	-23.8	17.2	-34.2	-27.0	20.4	-35.8				
Plane Frame Alternatives								(1.2	-3.4	3.4)				
	1	-41.6	27.1	-47.8	-26.8	17.4	-30.6	-28.0	20.8	-33.9	23.1	-20.9		
	2	-36.7	26.9	-53.4	-23.5	17.3	-34.3	-25.9	20.7	-36.3	23.1	-21.0		
	3	-41.7	27.1	-47.8	-26.8	17.4	-30.6	-28.0	20.8	-33.9	23.1	-20.9		
	4	-36.8	26.9	-53.4	-23.6	17.3	-34.3	-25.9	20.7	-36.3	23.2	-21.0		
	5	-36.8	26.9	-53.4	-23.6	17.3	-34.3	-26.9	20.7	-36.0				
	6										16.0	- 8.0		

Units: Moments in kip ft.

TABLE 12. STRESS RESULTANTS OF GIRDER 3, FLOOR 1

Analysis Alternative		LOADING											
		Dead Load		Full Live Load		Live Load Maxima		Sym. Lateral Load			Unsym. Lateral Load		
		P	M_y	P	M_y	P	M_y	P	M_x	M_y	P	M_x	M_y
Space Frame Alternatives	1	17.9	40.4	11.5	26.0	11.9	25.0	-2.8	22.8	-23.5	-3.2	6.4	-26.5
	2	17.5	35.7	11.2	22.9	11.8	23.1	-2.9	22.8	-23.8	-3.3	6.5	-26.8
	3	17.9	40.4	11.5	26.0	11.9	25.0	-2.8	22.8	-23.5	-3.2	22.9	-26.4
	4	17.9	40.4	11.5	26.0	11.9	25.0	-2.8	22.8	-23.5	-3.2	6.4	-26.5
	5	17.9	40.4	11.5	26.0	11.9	25.0	-2.8	22.8	-23.5	-3.2	6.4	-26.4
	6	17.9	40.4	11.5	26.0	11.9	25.0	-2.8	22.8	-23.5	-3.2	6.4	-26.5
	7	17.5	35.7	11.2	22.9	11.8	23.2	-2.9	22.7	-23.8	-3.3	6.5	-26.8
	8	17.5	35.7	11.2	22.9	11.8	23.1	-2.9	22.7	-23.8	-3.3	6.5	-26.8
	9	17.5	35.7	11.2	22.9	11.8	23.3	-2.9	22.7	-23.9	-3.3	6.5	-26.9
	10	17.5	35.7	11.2	22.9	11.8	23.7						
Plane Frame Alternatives	1	17.6	41.6	11.3	26.8	11.7	27.9	-2.8		-23.1			
	2	16.9	36.7	10.8	23.5	11.4	25.9	-2.8		-23.1			
	3	17.6	41.7	11.3	26.8	11.7	27.9	-2.8		-23.1			
	4	16.9	36.8	10.9	23.6	11.4	25.9	-2.8		-23.2			
	5	16.9	36.8	10.9	23.6	11.5	26.9						
	6							-1.5		-16.0			

Units: Axial loads in kips; Moments in kip ft.

TABLE 13. STRESS RESULTANTS OF COLUMN 4, STORY 1

Analysis Alternative		Displacement			
		Floor 4	Floor 3	Floor 2	Floor 1
Space Frame Alternatives	1	889.7	2111.4	2827.9	3231.2
	2	885.5	2095.8	2799.1	3187.8
	3	889.2	2111.0	2827.4	3230.8
	4	889.6	2111.0	2827.2	3230.3
	5	888.7	2110.4	2826.8	3230.3
	6	888.8	2110.6	2826.8	3230.3
	7	885.4	2095.5	2798.7	3187.1
	8	884.5	2095.0	2798.1	3187.0
	9	884.6	2095.1	2798.2	3187.0
Plane Frame Alternatives	1	1125.6	2547.4	3325.5	3729.2
	2	1119.2	2524.4	3283.5	3666.1
	3	1125.6	2547.6	3325.7	3729.2
	4	1119.4	2524.8	3283.7	3666.3
	6	553.1	948.3	1097.6	1147.4

Note: The values tabulated represent the displacements in inches times the modulus of elasticity of concrete in kips/sq. in.

TABLE 14. LATERAL X DISPLACEMENTS OF COLUMN 5

Analysis Alternative		Execution Time, Seconds
Space Frame Alternatives	1	325
	2	236
	3	232
	4	205
	5	174
	6	156
	7	166
	8	135
	9	129
	10	102
Plane Frame Alternatives	1	51
	2	50
	3	51
	4	50
	5	49
	6	37

TABLE 15. COMPUTER TIME FOR DIFFERENT ALTERNATIVES

Design Method	Analysis Alternative	Positive or Negative	DESIGN QUANTITIES 1	2	3	4	5	6	7	8	9	10	11	12
Working Stress	S.F.1	Positive	-4.8	-6.9	-12.4	-16.7	-9.1	-11.5	-19.5	-25.2	22.2	47.5	24.5	51.7
		Negative	-38.6	-40.9	-71.0	-81.2	-41.6	-46.0	-79.6	-92.5	8.5	17.0	9.7	19.4
	S.F.9	Positive	-3.9	-8.6	-7.5	-21.8	-8.4	-12.7	-15.9	-28.9	22.2	47.2	24.5	51.5
		Negative	-37.0	-42.8	-66.1	-89.1	-40.3	-47.5	-74.5	-98.3	8.5	16.8	9.7	19.3
	P.F.1	Positive	-5.3	-7.1	-13.0	-17.6	-9.5	-11.6	-20.1	-26.2	22.1	48.0	24.3	52.2
		Negative	-38.4	-40.6	-69.5	-81.7	-41.5	-45.8	-78.7	-93.1	8.5	16.9	9.7	19.3
Ultimate Strength	S.F.1	Positive	-3.6	-6.6	-11.2	-18.6	-9.4	-12.4	-20.8	-29.0	36.1	77.4	39.8	84.2
		Negative	-64.4	-68.1	-118.3	-131.9	-67.4	-74.6	-129.1	-150.1	11.1	23.1	12.5	25.9
	S.F.9	Positive	-2.9	-8.3	-6.3	-23.3	-8.9	-13.8	-17.3	-32.4	36.0	76.9	39.7	83.9
		Negative	-61.6	-70.2	-110.1	-144.4	-65.5	-76.9	-120.9	-159.2	11.1	22.9	12.5	25.8
	P.F.1	Positive	-3.9	-6.7	-12.0	-20.0	-9.6	-12.4	-21.7	-30.6	35.9	78.2	39.6	85.1
		Negative	-64.0	-67.6	-115.9	-132.7	-67.3	-74.2	-127.5	-151.1	11.1	23.2	12.5	26.0

Units: Moments in kip ft.

TABLE 16. DESIGN QUANTITIES OF SLAB 1, FLOOR 1

Design Method	Analysis Alternative	Column End	DESIGN QUANTITIES								
			Maximum P			Maximum M_x			Maximum M_y		
			P	M_x	M_y	P	M_x	M_y	P	M_x	M_y
Working Stress	S.F.1	B	29.8	0	65.4	19.9	30.2	57.6	24.4	-17.1	68.3
		A	34.7	0	31.7	28.5	16.3	37.4	34.0	0	50.1
	S.F.9	B	29.2	0	58.9	19.5	30.1	53.3	24.1	-17.0	63.7
		A	34.1	0	27.1	27.9	16.5	33.3	33.3	0	45.7
	P.F.1	B	30.1	0	69.5	29.2	28.5	69.5			
		A	34.9	0	46.1	33.4	15.1	51.1			
Ultimate Strength	S.F.1	B	48.3	0	105.6	33.1	50.4	96.1	40.7	-28.5	113.8
		A	55.6	0	48.6	44.4	29.3	58.8	54.4	0	81.6
	S.F.9	B	47.4	0	95.3	32.5	50.2	88.8	40.1	-28.4	106.1
		A	54.6	0	41.2	43.5	29.7	52.4	53.3	0	74.7
	P.F.1	B	48.7	0	112.6	40.0	47.4	115.9			
		A	55.9	0	74.3	53.5	27.1	83.3			

Units: Axial loads in kips; Moments in kip ft.

TABLE 17. DESIGN QUANTITIES OF COLUMN 4, STORY 1

Analysis Alternative	DESIGN METHOD	
	WORKING STRESS	ULTIMATE STRENGTH
S.F.1	.700 0 .800 0 .370 0 .199 0 .220 0 .226 0 .802* 0 .848 0+ .346 0	.609 0 .685 0 .305 0 .176 0 .191 0 .194 0 .705 0 .754 0 .284 0
S.F.9	.651 0 .878 0 .367 0 .187 0 .237 0 .225 0 .767 0 .893 0 .345 0	.564 0 .759 0 .303 0 .166 0 .203 0 .193 0 .672 0 .778 0 .283 0
P.F.1	.685 0 .805 0 .373 0 .196 0 .220 0 .227 0 .797 0 .842 0 .344 0	.596 0 .689 0 .308 0 .173 0 .191 0 .195 0 .701 0 .747 0 .282 0

* Top steel area   + Bottom steel area

Units: Steel areas in sq. in./ft. width of the slab

TABLE 18. REINFORCING STEEL IN LONGITUDINAL DIRECTION OF SLAB 1, FLOOR 1

Analysis Alternative	DESIGN METHOD					
	WORKING STRESS			ULTIMATE STRENGTH		
S.F.1	1.092 .455	.296 0	1.040 .321	.950 0	.240 0	.903 0
	0 .429	0 .294	0 .453	0 .354	0 .239	0 .375
	.981* .175+	.248 0	.887 0	.848 0	.211 0	.764 0
S.F.9	1.128 .551	.310 0	1.108 .499	.983 0	.253 0	.965 0
	0 .428	0 .293	0 .451	0 .353	0 .238	0 .374
	.950 .102	.236 0	.826 0	.821 0	.201 0	.708 0
P.F.1	1.085 .439	.297 0	1.047 .340	.944 0	.241 0	910 0
	0 .426	0 .294	0 .457	0 .352	0 .240	0 380
	.979 .171	.246 0	.872 0	.846 0	.210 0	754 0

* Top Steel

\+ Bottom Steel

Units: Steel areas in sq. in./ft. width of the slab.

TABLE 19. REINFORCING STEEL IN LATERAL DIRECTION OF SLAB 1, FLOOR 1

Column No.	Story	DESIGN METHOD											
		Working Stress						Ultimate Strength					
		S.F.1		S.F.9		P.F.1		S.F.1		S.F.9		P.F.1	
		t	A_{st}	t	A_{st}	t	A_{st}	t	A_{st}	t	A_{st}	t	A_{st}
1	1	20	18.72	20	15.24	20	18.72	19	12.48	18	15.24	18	15.24
	2	19	15.24	19	15.24	19	15.24	17	12.48	18	10.16	17	12.48
	3	21	18.72	20	18.72	20	18.72	18	15.24	18	15.24	18	12.48
	4	23	20.32	23	20.32	20	18.72	19	15.24	19	15.24	18	15.24
2	1	19	15.24	19	15.24	20	18.72	18	12.48	18	12.48	18	15.24
	2	20	15.24	20	15.24	20	18.72	17	12.48	17	12.48	18	12.48
	3	21	18.72	21	18.72	21	20.32	18	15.24	18	15.24	18	15.24
	4	21	20.32	21	20.32	22	20.32	19	15.24	19	15.24	19	15.24
4	1	19	15.24	19	15.24	20	15.24	18	12.48	17	12.48	18	15.24
	2	20	15.24	20	15.24	20	18.72	18	10.16	17	12.48	18	12.48
	3	21	18.72	21	18.72	21	20.32	18	15.24	18	15.24	19	15.24
	4	20	18.72	20	18.72	23	20.32	19	15.24	19	15.24	20	18.72
5	1	18	12.48	18	12.48	19	15.24	15	10.16	15	10.16	16	12.48
	2	20	15.24	20	15.24	21	18.72	17	12.48	17	12.48	18	15.24
	3	20	18.72	20	18.72	21	20.32	19	15.24	19	15.24	20	18.72
	4	21	18.72	21	18.72	22	20.32	20	18.72	20	18.72	21	18.72

Units: t in inches; A_{st} in sq. in.

TABLE 20. RESULTS OF DESIGN FOR COLUMNS 1,2,4&5

Design Method	Analysis Alternative	Quantities			
		Slabs		Columns	
		Concrete	Steel	Concrete	Steel
Working Stress	S.F.1	103.1	24416	50.4	28162
	S.F.9	103.1	24769	49.8	27598
	P.F.1	103.1	24288	50.2	29581
Ultimate Strength	S.F.1	103.1	17813	39.9	22122
	S.F.9	103.1	17815	39.4	22384
	P.F.1	103.1	17832	40.1	23355

Units: Concrete in cu. yds.; Steel in lbs.

TABLE 21. MATERIAL QUANTITIES

Columns	Story	DESIGN NO.					
		1		2		3	
		Trial t = 21''		Trial t = 12''		Trial t = 30''	
		t (5 cycles)	A_{st}	t (6 cycles)	A_{st}	t (7 cycles)	A_{st}
1,4	1	21	20.32	19	20.32	21	20.32
	2	14	8.00	16	12.48	14	8.00
	3	21	24.96	21	24.96	21	24.96
2,3	1	18	15.24	16	15.24	18	15.24
	2	18	18.72	18	18.72	19	20.32
	3	19	18.72	19	18.72	18	18.72
5,8	1	21	20.32	19	20.32	21	24.96
	2	21	24.96	18	18.72	17	15.24
	3	19	18.72	19	18.72	21	24.96
6,7	1	15	12.48	15	12.48	15	12.48
	2	19	20.32	20	20.32	19	20.32
	3	23	31.20	24	31.20	23	31.20
9,12	1	21	24.96	21	24.96	21	24.96
	2	18	18.72	18	18.72	18	18.72
	3	21	24.96	21	24.96	21	24.96
10,11	1	21	20.32	18	18.72	21	20.32
	2	21	24.96	21	24.96	21	24.96
	3	21	24.96	20	20.32	21	24.96

Units: t in inches; A_{st} in sq. in.

TABLE 22. RESULTS FOR COLUMNS, DESIGN NOS. 1,2 AND 3

Design No.	Longitudinal Direction	Lateral Direction
1	1.007* 0 1.226 0+ .445 0 .249 0 .318 0 .275 0 .879 0 1.158 0 .410 0	1.534 .382 1.348 .386 0 0 0 0 0 .646 .443 .685 1.871 .479 1.613 .611 0 .576
2	1.010 0 1.233 0 .446 0 .235 0 .324 0 .283 0 .789 0 1.193 0 .455 0	1.538 .377 1.310 .394 0 0 0 0 0 .653 .450 .698 1.790 .477 1.658 .507 0 .684
3	1.005 0 1.228 0 .447 0 .234 0 .322 0 .282 0 .787 0 1.184 0 .449 0	1.520 .382 1.360 .353 0 0 0 0 0 .651 .446 .686 1.774 .467 1.608 .487 0 .563

* Top Steel + Bottom Steel

Units: Steel areas in sq. in./ft. width of the slab.

TABLE 23. REINFORCING STEEL FOR SLAB 4, FLOOR 2 IN DESIGN NOS. 1, 2, 3

Design No.	Cycle No.	Concrete			Steel		
		Slabs	Columns	Total	Slabs	Columns	Total
1	1	245.3	56.3	301.6	43843	38015	81858
	5 (final)	245.3	52.9	298.2	44053	37683	81736
2	1	245.3	42.9	288.2	50964	30396	81360
	6 (final)	245.3	50.3	295.6	44749	36499	81248
3	1	245.3	77.2	322.5	40467	58077	98544
	7 (final)	245.3	52.4	297.7	43979	38030	82009

Units: Concrete in cu. yds.; Steel in lbs.

TABLE 24. MATERIAL QUANTITIES FOR DESIGN NOS. 1, 2 AND 3

Design Nos.	GROUPS OF SLABS OF EQUAL DEPTH			
	Group No.	Stories	Slabs	Depth, in.
4	1	1,2	All	10
	2	3	All	11
5,6	1	1,2,3	1,2,3	10
	2	1,2,3	4,5,6	11

TABLE 25. SLAB GROUPS AND SLAB DEPTHS FOR DESIGN NOS. 4, 5 AND 6

GROUPS OF COLUMNS OF EQUAL DIMENSIONS											
Group No.	Story No.	Trial t	Column Nos.	CYCLE 1				CYCLE 3 (Final)			
				Calculated		Revised		Calculated		Revised	
				t	A_{st}	t	A_{st}	t	A_{st}	t	A_{st}
1	1	21	1,4	20	20.32	21	18.72	20	20.32	21	18.72
			9,12	21*	24.96	21	24.96	21*	24.96	21	24.96
2	2	20	1,4	18*	18.72	18	18.72	17	15.24	18	15.24
			9,12	18	18.72	18	18.72	18*	15.24	18	15.24
3	3	21	1,4	21*	20.32	21	20.32	21*	24.96	21	24.96
			9,12	21	24.96	21	24.96	21	24.96	21	24.96
4	1	20	2,3	18	18.72	20	15.24	18	18.72	20	15.24
			5,8	19	18.72	20	18.72	19	20.32	20	18.72
			10,11	20*	20.32	20	20.32	20*	20.32	20	20.32
5	2	20	2,3	19	18.72	20	18.72	19	18.72	20	18.72
			5,8	19	18.72	20	18.72	20*	18.72	20	18.72
			10,11	20*	18.72	20	18.72	20	20.32	20	20.32
6	3	21	2,3	20	20.32	21	18.72	19	20.32	20	18.72
			5,8	20	20.32	21	18.72	19	20.32	20	18.72
			10,11	21*	24.96	21	24.96	20*	20.32	20	20.32
GROUPS OF IDENTICAL COLUMNS											
7	1	18	6,7	16	12.48	16	12.48	15	12.48	15	12.48
8	2	20	6,7	19	20.32	19	20.32	19	18.72	19	18.72
9	3	22	6,7	23	31.20	23	31.20	23	31.20	23	31.20

* Denotes control dimension of group

Units: t in inches; A_{st} in sq. in.

TABLE 26. COLUMN GROUPS AND RESULTS FOR COLUMNS, DESIGN NO. 4

GROUPS OF COLUMNS OF EQUAL DIMENSIONS											
Group No.	Trial t	Story No.	Column Nos.	Cycle 1				Cycle 2 (Final)			
				Calculated		Revised		Calculated		Revised	
				t	A_{st}	t	A_{st}	t	A_{st}	t	A_{st}
1	21	1	1,4	20	20.32	21	18.72	20	20.32	21	18.72
			9,12	21	24.96	21	24.96	21	24.96	21	24.96
		2	1,4	20	20.32	21	15.24	20	20.32	21	15.24
			9,12	20	20.32	21	18.72	20	20.32	21	18.72
		3	1,4	21*	24.96	21	24.96	21*	24.96	21	24.96
			9,12	21	24.96	21	24.96	21	24.96	21	24.96
2	20	1	2,3	19	20.32	20	12.48	20	12.48	20	12.48
			5,8	19	20.32	20	18.72	19	20.32	20	18.72
			10,11	20	18.72	20	18.72	20	18.72	20	18.72
		2	2,3	19	20.32	20	18.72	19	20.32	20	18.72
			5,8	19	20.32	20	18.72	19	20.32	20	18.72
			10,11	20	20.32	20	20.32	20	20.32	20	20.32
		3	2,3	19	20.32	20	18.72	19	20.32	20	18.72
			5,8	20*	18.72	20	18.72	20*	18.72	20	18.72
			10,11	20	20.32	20	20.32	20	20.32	20	20.32
3	22	1	6,7	16	12.48	23	10.16	15	12.48	23	10.16
		2	6,7	20	20.32	23	18.72	20	20.32	23	18.72
		3	6,7	23*	31.20	23	31.20	23*	31.20	23	31.20

* Denotes control dimension of group

Units: t in inches; A_{st} in sq. in.

TABLE 27. COLUMN GROUPS AND RESULTS FOR COLUMNS, DESIGN NO. 5

Column Nos.	Story No.	DESIGN NO. 4		DESIGN NO. 5		DESIGN NO. 6	
		t	A_{st}	t	A_{st}	t	A_{st}
1,4	1	21	18.72	21	18.72	21	18.72
	2	18	15.24	21	15.24	21	15.24
	3	21	24.96	21	24.96	21	20.32
2,3	1	20	15.24	20	12.48	21	15.24
	2	20	18.72	20	18.72	21	18.72
	3	20	18.72	20	18.72	21	18.72
5,8	1	20	18.72	20	18.72	21	18.72
	2	20	18.72	20	18.72	21	18.72
	3	20	18.72	20	18.72	21	18.72
6,7	1	15	12.48	23	10.16	23	10.16
	2	19	18.72	23	18.72	23	18.72
	3	23	31.20	23	31.20	23	31.20
9,12	1	21	24.96	21	24.96	21	24.96
	2	18	15.24	21	18.72	21	18.72
	3	21	24.96	21	24.96	21	24.96
10,11	1	20	20.32	20	18.72	21	20.32
	2	20	20.32	20	20.32	21	20.32
	3	20	20.32	20	20.32	21	18.72

Units: t in inches; A_{st} in sq. in.

TABLE 28. SUMMARY OF RESULTS FOR COLUMNS, DESIGN NOS. 4, 5 AND 6

Design No.	Concrete			Steel		
	Slabs	Columns	Total	Slabs	Columns	Total
1	245.3	52.9	298.2	44053	37683	81736
4	266.4	54.0	315.0	39279	35679	74958
5	266.4	59.1	325.5	38355	35612	73967
6	266.4	61.9	328.3	38355	34866	73221

Units: Concrete in cu. yds.; Steel in lbs.

TABLE 29. MATERIAL QUANTITIES FOR DESIGN NOS. 1, 4, 5 AND 6

Design Nos.	Group No.*	Slabs	Story	Identical Slabs	Depth, in.
7,8	1	1,2,3	1	1,3	10
			2	1,3	
			3	1,3	
	2	4,5,6	1	4,6	11
			2	4,6	
			3	4,6	

* Groups of slabs of equal depth.

TABLE 30. SLAB GROUPS AND SLAB DEPTHS FOR DESIGN NOS. 7 AND 8

Group[+] No.	Trial t	Story No.	Identical Columns	Control Columns	Calculated t	Calculated A_{st}	Revised t	Revised A_{st}
1	22	1	1,4,9,12	9	21*	24.96	21	24.96
		2	1,4,9,12	9	19	18.72	21	18.72
		3	1,4,9,12	9	19	18.72	21	18.72
2	22	1	2,3,5,8, 10,11	5 10	22 22	18.72 24.96*	22	24.96
		2	2,3,5,8, 10,11	5 10	21 22*	20.32* 20.32	22	20.32
		3	2,3,5,8, 10,11	5 10	21 21	24.96* 24.96	22	24.96
3	22	1	6,7	6	23	12.48	23	12.48
		2	6,7	6	23	18.72	23	18.72
		3	6,7	6	23*	31.20	23	31.20

[+] Groups of columns of equal dimensions.

* Denotes control dimension or control area of group.

Units: t in inches; A_{st} in sq. in.

TABLE 31. COLUMN GROUPS AND RESULTS FOR COLUMNS, DESIGN NO. 7

GROUPS OF IDENTICAL COLUMNS								
Group No.	Trial t	Identical Columns	Story No.	Control Column	Calculated t	Calculated A_{st}	Revised t	Revised A_{st}
1	20	2,3	1	2	22*	24.96	22	24.96
			2	2	20	18.72		
			3	2	19	18.72		
2	21	1,4,5 8,9,12	1	5	23*	24.96	23	24.96
			2	5	20	20.32		
			3	5	19	18.72		
3	23	6,7	1	6	23	24.96	23	31.20
			2	6	23	24.96		
			3	6	23*	31.20		
4	22	10,11	1	10	25*	37.44	25	37.44
			2	10	22	24.96		
			3	10	21	24.96		

* Denotes control column of group.

Units: t in inches; A_{st} in sq. in.

TABLE 32. COLUMN GROUPS AND RESULTS FOR COLUMNS, DESIGN NO. 8

Design No.	Concrete			Steel		
	Slabs	Columns	Total	Slabs	Columns	Total
5	266.4	59.1	325.5	38355	35612	73967
7	266.4	64.8	331.2	46325	39866	86191
8	266.4	73.0	339.4	65095	50450	115545

Units: Concrete in cu. yds.; Steel in lbs.

TABLE 33. MATERIAL QUANTITIES FOR DESIGN NOS. 7 AND 8

Design Output	Change in Data	Slabs				Columns	
		No. of Points concrete overstressed	Max. overstress %	No. of Points steel overstressed	Max. overstress %	No. Unsafe	Max. Unsafe %
PRELIMINARY	No change	8*	0.5	0	0	0	0
	f_{ca} = 1305 psi instead of 1350 psi	38	4.0	0	0	0	0
	Max. live load instead of full live load	32	5.0	302*	31.8	4	1.4
	Max. live load and f_{ca} = 1305 psi	62	8.6	302	31.8	4	1.4
FINAL	No change	0	0	0	0	0	0
	f_{ca} = 1305 psi instead of 1350 psi	42	3.4	0	0	0	0
	Max. live load instead of full live load	38	5.0	300	67.6	4	1.4
	Max. live load and f_{ca} = 1305 psi	68	8.6	300	67.6	4	1.4

* Out of 324 points.

TABLE 34. CHECKING OF DESIGN NO. 5

APPENDIX A

GENERATION OF TOPOLOGICAL INFORMATION

A.1 Numbering Scheme and Definition of Symbols

Since the processing of a structure is performed by stories, the same numbering scheme is applicable to the elements of every story in the structure. Figure A.1 shows the internal numbering scheme used for the elements of a typical story.

The following list defines the common symbols occurring in the program segments given in Art. A.2 and A.3:

NBAY = Number of bays

NBP = NBAY + 1

NAISLE = Number of aisles

NAP = NAISLE + 1

NG = Total number of girders (longitudinal plus lateral) in one floor of the space frame

NLONG = Number of longitudinal girders in one floor of the space frame

I = Story number (starting from the top of the structure).

A.2 Topological Relations for Space Frames

1) Given: Slab number, J

Required: Aisle number, KAISL, bay number, KBAY, and corner joint numbers, JOINT(K). The numbering of the joints at the corners of an individual slab is as shown in Fig. A.2(a).

Program Segment:

```
KAISL   = 1+(J-1)/NBAY
KBAY    = J-(KAISL-1)*NBAY
```

```
JOINT(1) = (KAISL-1)*NBP+KBAY
JOINT(2) = JOINT(1)+1
JOINT(3) = JOINT(1)+NBP
JOINT(4) = JOINT(3)+1
```

2) Given: Space frame joint number, J

Required: Longitudinal frame number, LFNO, and lateral frame number, LATFNO, intersecting at joint J.

Program Segment:

```
LFNO   = 1+(J-1)/NBP
LATFNO = J-(LFNO-1)*NBP
```

3) Given: Space frame joint number, J

Required: Space frame member numbers, MEMB(K), of the members framing into the joint. The sequence in which the members framing into the joint are considered and their direction is shown in Fig. A.2(b).

Determination: The following admixture of FORTRAN and non-FORTRAN statements describes the determination of MEMB(K):

```
MEMB(5) = NG+J
If LFNO > 1
MEMB(1) = NLONG+(LATFNO-1)*NAISLE+LFNO-1
If LATFNO > 1
MEMB(2) = (LFNO-1)*NBAY+LATFNO-1
If LATFNO < NBP
MEMB(3) = (LFNO-1)*NBAY+LATFNO
If LFNO < NAP
MEMB(4) = NLONG+(LATFNO-1)*NAISLE+LFNO
If I > 1
MEMB(6) = NG+J
```

4) Given: Slab number, J

Required: Space frame member numbers, LBEAM(K), of the girders along the edges of the slab. The numbering of the girders for an individual slab is as shown in Fig. A.2(a).

Program Segment:

```
LBEAM(1) = J
LBEAM(2) = J+NBAY
LBEAM(3) = NLONG+KAISL+(KBAY-1)*NAISLE
LBEAM(4) = LBEAM(3)+NAISLE
```

5) Given: Slab number, J

Required: The four adjacent slabs, NSL(K). The numbering of the adjacent slabs associated with Slab J is shown in Fig. A.2(c).

Determination:

```
If KAISL > 1
NSL(1) = J-NBAY
If KAISL < NAISLE
NSL(2) = J+NBAY
If KBAY > 1
NSL(3) = J-1
If KBAY < NBAY
NSL(4) = J+1
```

6) Given: Space frame joint number, J

Required: The neighboring joints, JT(K), which are numbered as shown in Fig. A.2(d).

Determination:

```
If LATFNO > 1
JT(1) = J-1
```

If LATFNO < NBP

JT(2) = J+1

If LFNO > 1

JT(3) = J-NBP

If LFNO < NAP

JT(4) = J+NBP

7) Given: Space frame joint number, J

Required: The four adjacent slabs, NSLB(K). The numbering of the slabs adjacent to joint J is shown in Fig. A.2(e).

Determination:

If LATFNO > 1 and LFNO > 1

NSLB(1) = (LFNO-2)*NBAY+LATFNO-1

If LATFNO > 1 and LFNO < NAP

NSLB(2) = (LFNO-1)*NBAY+LATFNO-1

If LATFNO < NBP and LFNO > 1

NSLB(3) = (LFNO-2)*NBAY+LATFNO

If LATFNO < NBP and LFNO < NAP

NSLB(4) = (LFNO-1)*NBAY+LATFNO

8) Given: Slab number, J

Required: The four frames, KFRAME(K), bordering the sides of Slab J. The numbering of the frames for an individual slab is as shown in Fig. A.2(f).

Program Segment:

```
C  LONGITUDINAL FRAMES
   KFRAME(1) = (J-1)/NBAY+1
   KFRAME(2) = KFRAME(1)+1
C  LATERAL FRAMES
```

KFRAME(3) = J-(KFRAME(1)-1)*NBAY

KFRAME(4) = KFRAME(3)+1

A.3 Topological Relations for Plane Frames

1) Given: Plane frame joint number, J

Required: Space frame member numbers, MEMB(K), of the plane frame members framing into the joint. The sequence in which the members are considered is shown in Fig. A.3(a).

Determination:

a) Longitudinal Frames (LFNO)

MEMB(3) = NG+(LFNO-1)*NBP+J

If J > 1

MEMB(1) = (LFNO-1)*NBAY+J-1

If J < NBP

MEMB(2) = (LFNO-1)*NBAY+J

If I > 1

MEMB(4) = MEMB(3)

b) Lateral Frames (LATFNO)

MEMB(3) = NG+(J-1)*NBP+LATFNO

If J > 1

MEMB(1) = NLONG+(LATFNO-1)*NAISLE+J-1

If J < NAP

MEMB(2) = NLONG+(LATFNO-1)*NAISLE+J

If I > 1

MEMB(4) = MEMB(3)

2) Given: Plane frame girder number, J

Required: The corresponding space frame girder number, MNO.

Program Segment:

a) Longitudinal Frames

MNO = (LFNO-1)*NBAY+J

b) Lateral Frames

MNO = NLONG+(LATFNO-1)*NAISLE+J

3) Given: Plane frame girder number, J

Required: The two adjacent slabs, NSL(K). The numbering of the slabs adjacent to girder J is as shown in Fig. A.3(b).

Determination:

a) Longitudinal Frames

If LFNO > 1

NSL(1) = (LFNO-2)*NBAY+J

If LFNO < NAP

NSL(2) = (LFNO-1)*NBAY+J

b) Lateral Frames

If LATFNO > 1

NSL(1) = (J-1)*NBAY+LATFNO-1

If LATFNO < NBP

NSL(2) = (J-1)*NBAY+LATFNO

4) Given: Plane frame joint number, J

Required: The four adjacent slabs, NSLB(K). The numbering of the slabs adjacent to joint J is shown in Fig. A.3(c).

Determination:

a) Longitudinal Frames

If J > 1 and LFNO > 1

NSLB(1) = (LFNO-2)*NBAY+J-1

If J > 1 and LFNO < NAP

NSLB(2) = (LFNO-1)*NBAY+J-1

If J < NBP and LFNO > 1

NSLB(3) = (LFNO-2)*NBAY+J

If J < NBP and LFNO < NAP

NSLB(4) = (LFNO-1)*NBAY+J

b) Lateral Frames

If J > 1 and LATFNO > 1

NSLB(1) = (J-2)*NBAY+LATFNO-1

If J > 1 and LATFNO < NBP

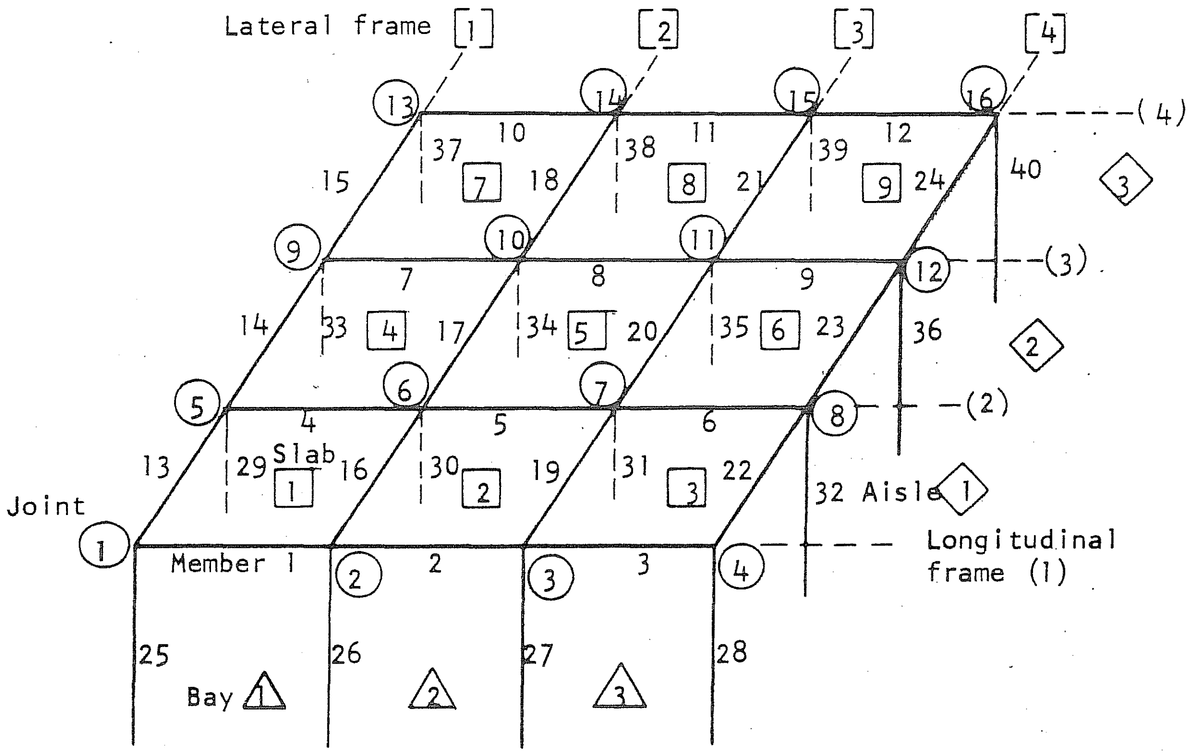
NSLB(2) = (J-2)*NBAY+LATFNO

If J < NAP and LATFNO > 1

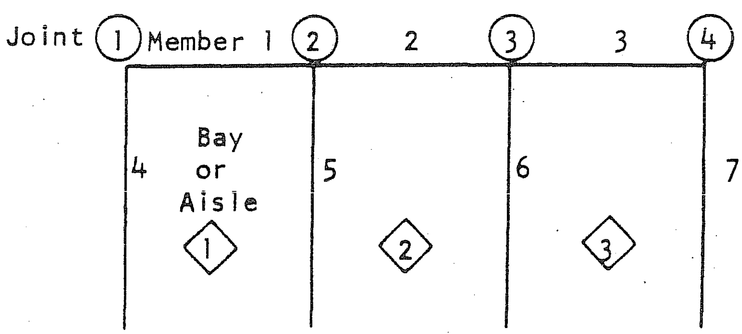
NSLB(3) = (J-1)*NBAY+LATFNO-1

If J < NAP and LATFNO < NBP

NSLB(4) = (J-1)*NBAY+LATFNO

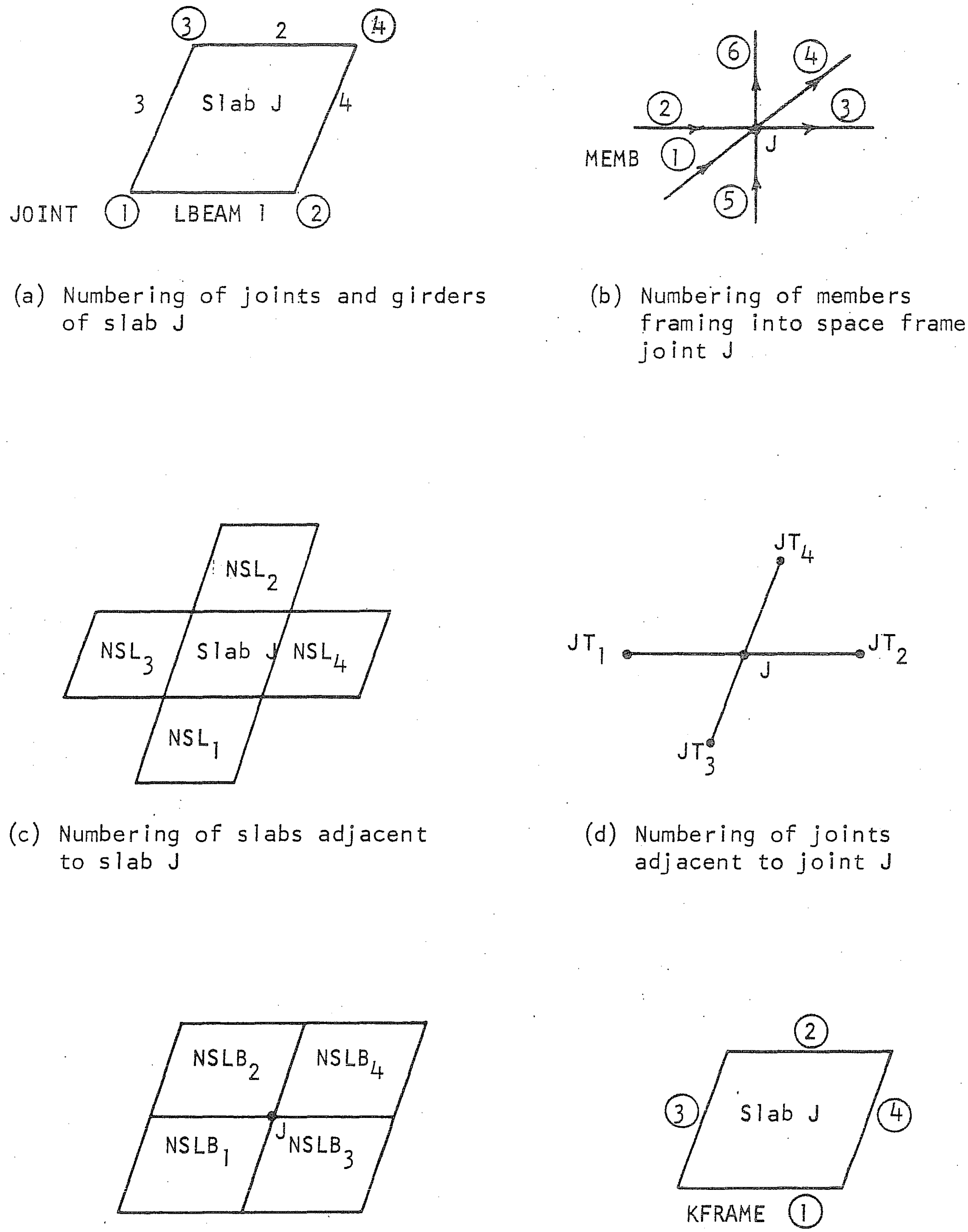


(a) Space Frames



(b) Plane Frames (Longitudinal or Lateral)

FIG. A.1 INTERNAL NUMBERING OF THE ELEMENTS OF A TYPICAL STORY



(a) Numbering of joints and girders of slab J

(b) Numbering of members framing into space frame joint J

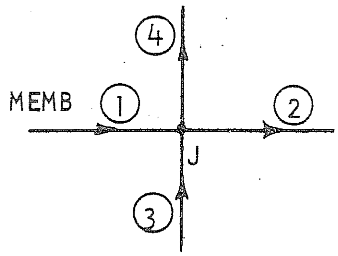
(c) Numbering of slabs adjacent to slab J

(d) Numbering of joints adjacent to joint J

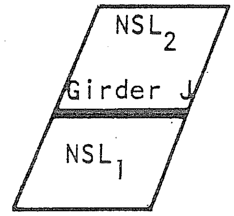
(e) Numbering of slabs adjacent to joint J

(f) Numbering of frames bordering slab J

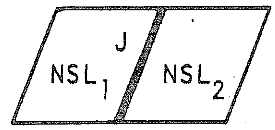
FIG. A.2 NUMBERING FOR SPACE FRAME TOPOLOGICAL RELATIONS



(a) Numbering of members framing into plane frame joint J

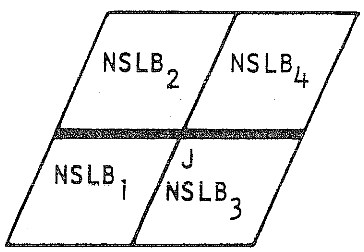


(i) Longitudinal frames

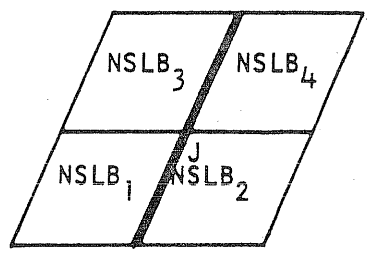


(ii) Lateral frames

(b) Numbering of slabs adjacent to girder J



(i) Longitudinal frames



(ii) Lateral frames

(c) Numbering of slabs adjacent to joint J

FIG. A.3 NUMBERING FOR PLANE FRAME TOPOLOGICAL RELATIONS

APPENDIX B

DESCRIPTION OF INPUT DATA

B.1 Introduction

The input data to the program are described below. The numeric control flags used to direct the program are described first. The remaining data are divided into blocks associated with the control statements described in Chapter 5. For the member group, loading and loading combination data, examples using problem-oriented statements are given for clarity. The equivalent numeric codes presently used are also given.

B.2 Numeric Control Flags

FORTRAN Name	Interpretation
KFLAG = 1	A new structure is being started.
= 2	A modification in the data has to be made.
= 3	Design or check the structure, depending upon the value of KDES.
= 4	Compute the material quantities for the structure.
KDES = 1	The system is being applied to design.
= 2	The system is being applied to checking.
LATLD = 0	The structure is not subject to lateral loads.
= 1	The structure is subject to lateral loads.
IUSD = 0	Design or Check by working stress method.
= 1	Design or check by ultimate strength method.

FORTRAN Name	Interpretation
When KDES = 1 only:	
KOUTPT = 0	Punched output for transmission to checker is not required.
= 1	Punched output for transmission to checker is required.

B.3 Data Associated with Control Words

B.3.1 Geometry

FORTRAN Name	Explanation
NBAY	Number of bays
NAISLE	Number of aisles
NSTORY	Number of stories
BW(I)	Width of bay I, ft.
AW(I)	Width of aisle I, ft.
SH(I)	Height of story I, ft.

B.3.2 Constants

FORTRAN Name	Explanation
FCPR	Compressive strength of concrete, psi.
FCA	Allowable stress in concrete, psi.
FY	Yield strength of reinforcing steel, psi.
FSA	Allowable stress in reinforcing steel, psi.
RMOD	Modular ratio, E_s/E_c.
POISS	Poisson's ratio for concrete.
UWC	Unit weight of concrete, lbs./cu. ft.
STCOV	Clear cover for top steel in slabs, in.

FORTRAN Name	Explanation
SBCOV	Clear cover for bottom steel in slabs, in.
CCOV	Clear cover for longitudinal steel in columns, in.
BPSR	Minimum allowable steel percentage for columns.
TPSR	Maximum allowable steel percentage for columns.
TRSP	Trial steel percentage for columns.

B.3.3 Analysis Data

FORTRAN Name	Explanation
LTYPE	= 1, for space frame analysis.
	= 2, for plane frame analysis.
NALT	Analysis alternative.
For plane frame analysis only:	
NLONGF	Number of longitudinal frames to be analyzed.
NLATF	Number of lateral frames to be analyzed.
LFR(I)	Frame numbers of longitudinal and lateral frames to be analyzed.

B.3.4 Groups

a) Problem-Oriented Statements:

SLAB GROUPS

GROUPS OF FIXED DEPTH

GROUP 1 DEPTH 8 IN.

FLOOR 1 SLABS 1, 5

FLOOR 2 SLABS 5, 6

GROUPS OF SAME DEPTH OR IDENTICAL SLABS

GROUP 1 SAME DEPTH

FLOOR 1 SLABS 2, 3

GROUP 2 SAME DEPTH

FLOOR 2 SLABS 2, 3

GROUP 3 IDENTICAL

FLOOR 1 SLABS 2, 4

FLOOR 4 SLABS 2, 4

COLUMN GROUPS

PREASSIGNED GROUPS

GROUP 1 EQUAL DIMENSIONS SIDE DIMENSION 20 IN.

STORY 1 COLUMNS 1, 3, 7, 9

STORY 2 COLUMNS 1, 3, 7, 9

GROUP 2 IDENTICAL SIDE DIMENSION 22 IN.

STORY 2 COLUMNS 1, 2, 4, 9

PROGRAM SELECTED GROUPS

EQUAL DIMENSIONS

COLUMNS BETWEEN 350 AND 420 SQ. IN.

COLUMNS BETWEEN 421 AND 470 SQ. IN.

IDENTICAL

COLUMNS BETWEEN 200 AND 260 SQ. IN.

SIDE DIMENSION OF OTHER COLUMNS 18 IN.

b) Dictionary for Transition from Problem-Oriented Statements to Numeric Input Code:

i) Slab Groups

Problem-Oriented Statement	Equivalent Statement
GROUPS OF FIXED DEPTH	GROUPS OF CATEGORY 1
GROUPS OF SAME DEPTH OR IDENTICAL SLABS	GROUPS OF CATEGORY 2
SAME DEPTH	TYPE 1
IDENTICAL	TYPE 2

ii) Column Groups

Problem-Oriented Statement	Equivalent Statement
PREASSIGNED GROUPS	GROUPS OF CATEGORY 1
PROGRAM SELECTED GROUPS	GROUPS OF CATEGORY 2
EQUAL DIMENSIONS	TYPE 1
IDENTICAL	TYPE 2

c) Numeric code in present version:

Problem-Oriented Words	Numeric Code
SLAB GROUPS	800
COLUMN GROUPS	801
GROUPS OF CATEGORY	802
GROUP	803
TYPE or DEPTH	804
FLOOR or STORY	805
SLABS or COLUMNS	806
COLUMNS BETWEEN	807
AND	808
SIDE DIMENSION	809
SIDE DIMENSION OF OTHER COLUMNS	810

B.3.5 Loads

a) Problem-Oriented Statements:

```
SUPERIMPOSED DEAD LOAD*
FLOOR 1
PANEL 1  0.10
PANEL 3  0.12
   :
FLOOR 2
PANEL 1  0.12
   :
JOINT LOADS
STORY 1
JOINT 1  FORCE X 10.0
JOINT 3  MOMENT Y 40.0
   :
```

* The format for live loads and arbitrary fixed panel loads is exactly the same.

b) Dictionary for load types:

Problem-Oriented Statement	Numeric Code
SUPERIMPOSED DEAD LOAD	1
LIVE LOAD	2
ARBITRARY FIXED PANEL LOADS	3
JOINT LOADS	4

c) Numeric code in present version:

Problem-Oriented Word	Numeric Code
FLOOR J or STORY J	100+J
PANEL J	200+J
JOINT J	300+J
FORCE or MOMENT	*

* The joint load directions FORCE X, FORCE Y, FORCE Z, MOMENT X, MOMENT Y and MOMENT Z are denoted by codes 1 through 6.

B.3.6 Load Combinations

a) Problem-Oriented Statements:

```
COMBINATION 1
LOADING 1   FACTOR 1.0
LOADING 3   FACTOR 0.75
LOADING 2   REVERSIBLE FACTOR 0.9 OR LOADING 5 FACTOR 1.0
:
COMBINATION 2
LOADING 1   FACTOR 0.75
LOADING 2   REVERSIBLE FACTOR 1.0
LOADING 4   FACTOR 0.9 OR LOADING 5 FACTOR 1.0
:
```

b) Numeric code in present version:

Problem-Oriented Word	Numeric Code
COMBINATION	900
LOADING	901
FACTOR	902
OR	903
REVERSIBLE	*

* Reversible loadings are denoted by a negative loading number.

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13. ABSTRACT

A computer-aided system for the analysis, design and checking of flat plate reinforced concrete buildings is presented, which allows for the engineer's participation at all the decision-making stages during the design process. The geometry of the structure is assumed to be regular, and the slabs are idealized as girders of one panel width each. The structure is analyzed either as a space frame or as a series of plane frames, and several levels of accuracy are included within the space and plane frame analyses. A general loading combination procedure is implemented and the members may be designed by either working stress or ultimate strength design methods in accordance with the ACI specifications. In addition, member groups may be specified and the structure may be designed on the basis of a partial analysis of the structure. The system is also applicable to the checking of a designed structure and to the computation of material quantities.

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